ASSESSMENT OF INSITU CONCRETE STRENGTH

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A B S T R A C T

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Assessment of the strength of insitu concrete is a subject area in which there have been significant developments in recent years. This is due to an increased awareness by Engineers of the limitations of previously existing test methods coupled with recognition of the benefits to be gained from a reliable knowledge of insitu strength. Developments which are considered in detail have included increased understanding and application of established testing methods, increased understanding of the characteristics of insitu concrete, and new testing methods aimed specifically at strength measurement.

The Author has contributed to these developments in the areas of testing of small diameter cores, ultrasonic pulse velocity measurements, penetration resistance, internal fracture testing and pull-out testing as well as the general philosophy of planning of investigations and interpretation of results. Laboratory investigations upon which these contributions are based are described in detail in the published papers which form the basis of this submission. The data and findings from these investigations have subsequently been used by Engineers in the course of field testing programmes and by other research workers.

The British Standard dealing with non-destructive methods of testing concrete is currently undergoing major revision and extension to include recently developed test methods for which the Author has produced data. Other aspects of the Author's work relating to the influence of reinforcement upon ultrasonic pulse velocity measurements and planning of investigations have been incorporated directly into the Drafts for public comment due to be published shortly.

Although the accuracy with which insitu strength can be assessed is limited there are many situations in which carefully planned and executed insitu testing may be most worthwhile if the results are properly interpreted. Use of such methods in practice has increased considerably over the past 15 years, and will continue to increase as confidence in their validity is established.

A C K N O W L E D G E M E N T S

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NOTES ON PRESENTATION

This submission consists of published papers by the Author preceeded by a statement which summarises the relationship of the work of the Author to the general body of knowledge in the subject. The statement also identifies the aims of the investigations which have led to these publications and discusses the principal results achieved. Papers by the Author are identified in the statement by the sequence 1 to 20 (and underlined), whilst general references are numbered from 101 to 122.

C O N T E N T S

Author's Papers

ASSESSMENT OF INSITU CONCRETE STRENGTH

1. INTRODUCTION

The assessment of the properties of insitu concrete is a problem with which Engineers have been concerned for many years. Most designs and materials specifications are notionally based upon the strength of concrete and it is therefore natural that this should be the property which is often considered to be of the greatest importance. The strength itself is frequently not the most critical property of the material, but nevertheless often provides a valuable indication of other significant characteristics.

Reported attempts were made in many parts of the world in the 1930's to develop reliable insitu strength measurement tests, but these were of limited success and were not fully developed. Following the Second World War two particular non-destructive testing techniques were developed (Rebound Hammer and Ultrasonic Pulse Velocity). These were unfortunately found to have complex correlations with concrete strength and thus gained only limited acceptance compared with the compression testing of cores cut from the concrete. Developments were slow, and Elvery (101) summarised the state of knowledge in 1973 relating to each of these three approaches, (Rebound Hammer, Ultrasonic Pulse Velocity and Cores). All were shown to have deficiencies, but Ultrasonic Pulse Velocity testing was identified as the more worthwhile of the two non-destructive approaches because it gave information about the state of concrete throughout the width or depth of a structural member, whereas the rebound method indicated only the state of the concrete near the surface.

In the 1970's a number of factors in the United Kingdom particularly served to highlight the inadequacies of the existing techniques. These factors included:

a) The need for strength assessment of large numbers of High Alumina Cement Concrete members as a result of suspected material deterioration.

b) A major increase in the number of relatively young concrete structures showing deterioration. This was attributed to changes in cement manufacturing processes coupled with a decline in standards of workmanship and supervision.

c) An increased interest in the advantages of early removal of formwork as a result of economic pressures.

Subsequently, there was an upsurge of interest in many countries in the development of new testing techniques aimed specifically at assessing the strength of insitu concrete, whilst efforts were also made to understand and overcome some of the difficulties associated with the more established methods., This has been coupled with work to improve understanding and general awareness of the characteristics of insitu concrete in relation to member type and practical circumstances to assist the interpretation and acceptance of the results of insitu testing. B.S. 6089 (102) entitled, "Guide to Assessment of concrete strength in existing structures" was published in 1981 and reflected some of the results achieved. Developments have been rapid and recently published technical papers indicate further significant advances in the past few years. Many of the advances in testing techniques and the interpretation of their results are currently being incorporated into revisions of B.S. 1881, "Methods of Testing Concrete" which are due for publication within the next two years.

2. THE NEED FOR INSITU STRENGTH ASSESSMENT

Information about the insitu strength of concrete may be required for both newly cast and old concrete. For new concrete the most commonly occurring circumstances include:

a) Non-compliance of the material supplied in terms of works cube test results or other specified requirements.

b) Uncertainties concerning the level of workmanship involved in construction operations affecting the hardened properties of the insitu concrete.

c) Quality Control of construction or manufacture.

d) Monitoring of strength development in relation to formwork removal, cessation of curing, prestressing, load application or similar purposes.

Older concrete may need to be examined when:

a) There is suspected deterioration of the concrete due to such factors as external or internal chemical attack or change, fire, explosion or other environmental effects.

b) An assessment is to be made of the load carrying capacity of an established structure for change of ownership or insurance purposes, or in relation to a proposed change of use or alteration.

3. AIMS OF THE AUTHOR'S INVESTIGATION

In the light of the inadequacies of the testing methods which were available in the early 1970's related to practical aspects of the assessment of insitu concrete strength, a programme of experimental investigations has been pursued by the Author with the following aims:

a) To examine aspects of Core testing and Ultrasonic Pulse Velocity testing which are of particular practical relevance when dealing with insitu concrete.

b) To appraise, modify as necessary, and develop the appropriate application of newly emerging test methods which attempt to provide a direct measure of concrete strength.

c) To gather data which would enable improved understanding of the problems associated with the assessment of insitu concrete strength and aid interpretation of the results of insitu testing.

This programme has been arranged so that these three aims were pursued simultaneously, as far as possible, and has also been undertaken within the framework of other work of a similar nature known to be in progress in the United Kingdom. It has thus not been exhaustive but, when considered in conjunction with other work published in the past 10 years, has enabled significantly greater confidence to be placed upon estimates of insitu concrete strength.

4. TESTING METHODS

4.1 CORES

The compression testing of cores provides the most direct method of assessment of concrete strength.

4.1.1 'Standard' Interpretation Procedures

British Standard B.S. 1881, Part 4: 1970, which has now been superceded by B.S. 1881, Pt. 120: 1983 (103), provided a simple correction procedure for test results. This made an adjustment to the measured strength for the length/diameter ratio of the cored specimen, followed by conversion to an equivalent cube strength based on an average factor of 1.25 between the strength of a cube and that of a standard cylinder. The shortcomings of this procedure were discussed in detail by the Concrete Society Technical Report No. 11 (104) in 1976 which introduced the terms 'Actual' and 'Potential' cube strength to describe the values developed from core tests. In particular, it was proposed that allowance be made for orientation of drilling, voidage ratio, and embedded reinforcement in estimating cube strengths. The proposals of that Report are discussed and illustrated in Paper 1 and compared with the then established British Standard approach. Detailed examination of the formulae proposed indicates a significant change in the values used to allow for variations of length/diameter ratio. This effect is illustrated in Figure 5.5 of Paper 1 whilst the overall relationships between estimates of cube strength based on the various procedures are presented in Figure 5.6 which highlights the difficulties of interpretation and application of results. Estimation of the Potential strength involves many uncertainties requiring considerable skill and experience to reliably assess and must thus be treated with caution. Most of these developments have now been incorporated into B.S. 6089 (102) and B.S. 1881 (103), but with the recommendation that results be presented only as "estimated insitu cube strength", i.e. 'Actual' cube strength.

4.1.2 Small Cores

The recommendations of B.S. 1881 (103) relate only to cores with a diameter of 100mm or over, although the use of cores of that size often presents many practical problems of damage, cutting effort and lack of spread of test locations. In some cases member size limitations may totally preclude the cutting of a suitably proportioned core of 100mm diameter. "Small Cores" with lesser diameters have thus often been used despite the limited available guidance concerning the interpretation of their results. The use of cores of 44mm diameter, which is a commonly used size and is about the smallest practicable size for compression testing, is examined in detail in Paper 2. This is the only English language publication which offers detailed information about the behaviour of small diameter cores containing a range of aggregate sizes, and forms the basis of interpretation of results of both practical investigations and experimental research. The work confirms the increased scatter to be expected from results in comparison with 'standard' cores and also identifies the effect of aggregate size upon both variability of results and measured strength. It is shown that the effects of length/diameter ratio are close to those proposed by the Concrete Society (104) for 'standard' cores, but that the relationship between measured core strength and corresponding actual cube strength is affected by the aggregate size as well as the orientation of drilling. The accepted procedures for interpretation of 'standard' core results therefore cannot necessarily be applied to smaller size specimens. In addition to providing general information and guidance about the behaviour of 'small' cores the detailed recommendations in Paper 2 enable cube strengths to be estimated and also permit an assessment of the likely accuracy of such estimations.

4.2 NON-DESTRUCTIVE METHODS

These are defined as methods which leave no physical damage to the concrete, but in practice may nevertheless cause surface marking or staining.

4.2.1 Rebound Measurements

This approach suffers the major disadvantage that results reflect only the surface properties of the concrete, and are thus greatly affected by ageing effects. Calibrations between rebound number and concrete strength are influenced by a large number of factors associated with both the concrete mix and practical features of the member under test.

Realistic estimates of insitu strength are not possible without the development of specific calibrations and are therefore confined to preplanned situations with newly cast concrete. The method is of greater value for comparative strength estimation purposes, but is again usually restricted to young concrete. In view of these restrictions, development of the method has received little attention in recent years, although the current situation is discussed in detail in Paper 3.

4.2.2 Ultrasonic Pulse Velocity Measurements

Developments in Ultrasonic Pulse Velocity measuring equipment in the past 10 years have been relatively minor. The most significant advances have been made in terms of interpretation and application of results for insitu concrete members. These have been accompanied by an increased general awareness of the capabilities and limitations of the method. The fundamental theory of the approach has been presented in detail by Jones (105) in 1962, and is summarised in Paper 4 together with a description of detailed procedures and test equipment now in common use. The relationship between pulse velocity and compressive strength is described in section 3.3.1 of Paper 4. Work by the Author has concentrated upon a number of aspects for which available information has been shown to be inadequate by practical experiences with the use of this technique to assess insitu concrete strength.

4.2.2.1 High Alumina Cement Concrete

Having been identified by Elvery (101) as the most worthwhile non-destructive approach to concrete strength estimation, this method received considerable attention when Engineers were faced with the need to examine the condition of very large numbers of suspect prestressed High Alumina Cement concrete members in the mid 1970's. The problems of concrete strength reduction as a result of conversion have subsequently been discussed in detail (106) but in 1974, despite earlier warnings of the potential problem (107), little firm information was available to guide Engineers undertaking inspections.

Following meetings with precast concrete manufacturers, local Engineers and a working group set up by the President of the Institution of Structural Engineers, the Author published the results of a limited laboratory investigation to examine pulse velocity/strength relationships for concretes made with High Alumina Cement. These are given in Paper 5 and confirm that for newly cast concrete the relationships are of the same general form as to be expected for Portland Cement concretes. It was found however that when the conversion process was accelerated under laboratory conditions, the reduction in concrete strength was not necessarily accompanied by a corresponding reduction in pulse velocity. This finding, which was confirmed by Mayfield (108) in a simultaneous investigation, cast some doubt on the value of pulse velocity measurements to assess the strength of converted concrete.

Pulse velocity measurements were, however, also taken for cores cut from a range of naturally converted concrete members from existing structures. These were compared with equivalent cube strengths derived from the cores using the British Standard procedure current at that time. The results are shown in Figure 5 of Paper 5, and although the scatter is considerable the general trends are clear. Paper 5 provided the first published detailed practical guidance concerning the use of ultrasonic pulse velocity measurements to assess High Alumina Cement concrete members in structures. It was suggested that the method could be used to indicate a likely strength range for naturally converted concrete but not to yield a precise strength estimate. The use of the technique to compare similar members to locate suspect areas and hence enable the most effective use of core or load tests was emphasised. Particular attention was also paid to the need to alert Engineers to the practical difficulties involved, and the level of skill necessary to achieve meaningful results from insitu investigations.

Ultrasonic testing was subsequently used widely in the course of High Alumina Cement Concrete investigations despite doubts about the reliability of strength estimations for such an unpredictable material, as indicated by Cusens and Jackson (109) for example. Further data were, however, presented by the Author in 1976, in Paper 6. Strength estimates from ultrasonic pulse velocity measurements are compared in Figure 6 with strengths calculated from flexural load test results on beams removed from structures. These results tend to confirm the Author's earlier view that the method may be used as a worthwhile indicator of concrete strength range in such circumstances.

4.2.2.2 Prestressed Concrete

Following early work in 1974, a number of practical features associated with insitu ultrasonic pulse velocity testing of small section prestressed concrete members were examined in greater detail by the Author and these findings are outlined in Paper 6. Particular aspects for which available information was limited were examined in the laboratory. These include the effects of short path length (Figure 3), the effect of transducer overhang on measurements taken across narrow flanges (Table 1), and the effects of compressive stress.

Preliminary comparisons were made between results for cubes and prisms under uniaxial compression, and measurements across the compressive zone of a beam subjected to flexure. These are illustrated in Figure 2 and suggest that pulse velocities in beams and prisms may be reduced due to microcracking at relatively lower levels of stress than may be expected for cubes. The influence of flexural compressive stress at the levels usually encountered under service conditions is likely to be negligible. Overstressing, possibly due to a reduction in concrete strength, may however lead to a reduction in measured pulse velocity thus enhancing the ability of Ultrasonic Pulse Velocity measurements to detect suspect situations. The investigations into the effects of compressive stress were extended in greater detail, and further results are presented in Table 2 of Paper 7. These later results indicate considerable variations in behaviour, and that the differences between cubes and beams may not be as great as previously suggested, but confirm that under normal service conditions pulse velocities are unlikely to be affected by compressive stress either in reinforced or prestressed beams.

4.2.2.3 Variability of Test Results

Considerable experience in the use of ultrasonic pulse velocity measurements to assess insitu concrete was gained during the late 1970's and this is reflected in Paper 4 and Paper 7. The influence of moisture conditions upon measured values, and hence strength correlations, was considered in detail by Tomsett (110) who also indicated that the variability of test results may reflect the standard of control exercised over construction. Results of tests by the Author on laboratory made reinforced concrete beams are presented in Paper 7 which demonstrate the variability of concrete properties which may be expected within such beams. The value of the use of Coefficient of Variation of pulse velocity measurements as an indicator of concrete strength uniformity is also illustrated by Table 3 of that Paper.

4.2.2.4 Effects of Reinforcement

The velocity of ultrasonic pulses through steel will usually be greater than through concrete. Embedded reinforcement or prestressing steel may therefore influence measurements made on concrete members if they are located close to the direct pulse path. Correction factors to make allowance for this effect are provided in B.S. 4408, Pt. 5 (111) and RILEM document N.D.T.1 (112). These factors, which are similar, do not take bar diameter into account and are stated to represent the maximum likely influence of reinforcement upon readings.

It was clear from preliminary laboratory tests by the Author upon specimens with embedded 6.3mm prestressing wire, as commonly found in small section pretensioned beams, that this steel has no detectable influence upon measured pulse velocities whether running along or across the pulse path. Since a small change in pulse velocity may be associated with a substantial change in concrete strength, the use of false correction factors may lead to serious misinterpretation of test results. The BSI (111) and RILEM (112) recommendations have been based largely upon theoretical analysis, with little published experimental data available. It was also clear from discussions with Engineers experienced in the use of Ultrasonic techniques that concern about the use of the available corrections was shared. An extensive programme of laboratory testing was therefore commenced by the Author to examine this problem in greater detail.

Attention was initially concentrated on bars running transversely across the pulse path since this is the most commonly occurring situation in which some allowance for reinforcement may be necessary. In 1978, Chung (113) presented experimental results for bars running along the line of the direct pulse path which clearly indicated the effect of bar diameter. Correction factors developed from Chung's work are compared with the established corrections in Figure 4 of Paper 7 and also in Figure 3.15 of Paper 4 for typical cases. Initial comparisons with work by the Author on specimens containing bars of 20mm diameter and greater indicated approximate agreement with Chung. The Author's results for transverse bars displayed a roughly similar pattern with varying diameter, but with a reduced magnitude of steel influence as indicated in Paper 7.

Further investigations involving bars of less than 20mm diameter, which could barely be detected when located across the pulse path, demonstrated clearly that Chung had underestimated the influence of bars in the 6mm to 12.5mm diameter range when running along the pulse path. This is discussed in detail in Paper 8 and illustrated by Figure 5 in particular. Paper 8 also presents in sections 4.2 and 4.3 preliminary information upon which more realistic correction factors may be based, together with information about the effects of practical factors such as bond and cracking. This work has finally been extended to provide a detailed comprehensive correction procedure for embedded reinforcing bars which is presented in Paper 9 and has been incorporated into the Draft of B.S. 1881, Pt. 203 (which will supercede B.S. 4408, Pt. 5) due for issue for public comment in the summer of 1984. This approach, developed by the Author, offers a more flexible and reliable method of allowing for reinforcement than previously available. As well as taking account of bar diameter in relation to the concrete properties (Figures 7 and 9 of Paper 9) the importance of end cover thickness is indicated by Figure 2 when small offset distances are involved. Typical comparisons with corrections based on other methods are shown in Figures 10 and 11, whilst the significance of the discrepancies is discussed in the text of that Paper.

4.3 DIRECT INSITU STRENGTH MEASUREMENT TESTS

A number of completely new tests have been developed in recent years. These include methods which are based on the concepts of pulling an insert from the concrete surface, pulling a probe attached to the surface, or breaking the concrete below the surface. A technique based on the penetration of a probe fired at the concrete surface has also been the subject of further research during this period. These methods which are summarised in Paper 10, and examined in greater detail in Paper 11, are characterised by the common feature of attempting to provide a direct measurement of a concrete strength property.

The actual property measured varies according to method, and in some cases is not clearly defined, but calibrations with compressive strength are subject to significantly fewer variables than the indirect methods. The methods are also similar in that they only provide an indication of concrete properties near to the surface and cause a limited amount of surface damage which may need to be made good. Consequently these approaches are often classified as Near to Surface methods. The term Partially Destructive is also often applied to these tests, although they are effectively non-destructive in relation to the body of a concrete member and its continued fitness for use.

Work by the Author has contributed to the general increase in knowledge and experience relating to a number of these techniques which have now gained a level of acceptance such that it is proposed that they are to be included in B.S. 1881, Pt. 207, 'Near to Surface Methods' for which a Draft is currently under preparation. Outline descriptions of the methods and their applications are also included in the Draft of B.S. 1881, Pt. 201, 'Guide to the use of non-destructive methods of test for concrete' which is scheduled for issue for public comment in June 1984.

4.3.1 Windsor Probe Test

Developed in the U.S.A. in the 1960's, this measures the depth of penetration of a steel bolt or 'probe' fired at the concrete surface. The technique is described in detail in Paper 12. Although available for some years the method has gained acceptance only slowly, and published data are limited. Principal applications in the U.S.A. have been related to quality control and strength development monitoring.

The Manufacturers of the test system suggest that correlation with compressive strength is influenced only by the hardness of the aggregate and present tables based on this assumption. In the course of investigations by the Author to examine the reliability of the method it became clear that such tables, which were based on North American aggregates, were of little relevance to gravels typical of those used in the United Kingdom. This is clearly illustrated by Figures 3 and 4 of Paper 12, which provided the first readily available data relating to typical British aggregates. The importance of aggregate type has been subsequently confirmed by Keiller (114).

The importance of specific calibrations for the particular aggregate in use to enable realistic strength estimations to be made is emphasised in Paper 12. It is also concluded that the most worthwhile applications will be of a comparative nature. The Author has subsequently found the test to be of particular value in comparative situations where access is difficult. The state of knowledge about the actions influencing the resistance to penetration of the concrete surface is such that interpretation of results can only, at the present time, be based upon empirical findings. This is likely to remain the case unless theoretical work of a fundamental nature is undertaken.

4.3.2 Pull-out Methods

These involve the use of cast-in inserts, which require preplanning, and devices inserted into surface drilled holes. Their development is discussed in Paper 11 whilst an up-to-date review of current knowledge is provided by Paper 13.

4.3.2.1 Internal Fracture Test

This test involves a 6mm diameter expanding wedge anchor bolt inserted into a hole drilled about 35mm below the concrete surface. The original form proposed by the Building Research Establishment requires the use of a torquemeter to apply loading to the bolt, but the Author has demonstrated in Paper 14 and Paper 15 that the high testing variability can be significantly reduced by the use of direct pull loading. Results of an extensive laboratory programme are presented in Paper 14 which demonstrate the importance of standardisation of loading rate and method of application. A simple mechanical direct pull equipment developed by the Author is described in Paper 14 and illustrated in Figure 5 of that paper. This was subsequently refined, and the version shown in Figure 1 of Paper 15 has provided the model for apparatus known to be in use by other investigators.

A further important feature of the results presented in Paper 14 is the discrepancy between the calibration curves for Torque and Compressive Strength obtained by the Author and those published by the Building Research Establishment. This is illustrated by Figures 6 and 7, is further considered in the Discussion to that Paper, and has since formed the basis of continuing dialogue between the Author and the B.R.E..

The Author's findings have been confirmed by others including Long and Glass (115), and more recently, Keiller (114). Recent tests with the torquemeter approach, using a modified component within the loading tripod, were performed by the Author in the presence of a B.R.E. representative. This is discussed in Paper 13 and illustrated in Figure 11 from which it can be seen that the effect is minor and that the Author's earlier findings are essentially confirmed. Attempts have been made to develop a theoretical model for the failure mechanism, as described in Papers 11 and 14, but this is of little value in resolving such disagreements.

The influence of compressive and tensile stresses within the concrete upon the results of tests of this type are also considered in Paper 14 on the basis of tests on substantial laboratory made reinforced concrete beams. Results, summarised in Table 7, indicate an appreciably greater scatter of values than to be expected on unstressed concrete. Table 7 of Paper 14 illustrates clearly the effects of concrete strength variation across the member depth, but the overall influence of compressive stress does not agree particularly well with the statements later made by Dr. Chabowski in the Discussion to Paper 14 and in Paper 15. It is clear that this is an area requiring further attention.

4.3.2.2 Lok-Test and Capo Test

Based upon a 25mm diameter insert at a depth of 25mm below the concrete surface these two tests represent cast-in and drilled versions of the same approach which are both commercially available from the Danish originators of the method. Development of the approach is described in Paper 11 and whilst the Lok-test has been used extensively in the U.S.A. and Scandinavia, experience in the United Kingdom is very limited. The results presented in Figures 8, 9 and 10 of Paper 13 are the only available published data relating to work carried out in Great Britain. These basically confirm the validity of the general calibration relationships proposed by the Manufacturer of the equipment for the range of aggregate types and conditions examined. The high within-test scatter of results is indicated in Figure 8, emphasising the need to average at least six individual values, and estimates are made of the accuracy of insitu strength prediction likely to be achieved. Although the accuracy of insitu strength prediction is not significantly better than by other methods, the availability of a reliable general calibration relationship is of considerable value. Extensive efforts have been made in Denmark to provide theoretical evidence to support the notion that failure is due to crushing of a compressive "strut" between the embedded disk and the reaction ring, as described in Paper 11. Recent work from the U.S.A. (116), however, casts doubt upon the validity of this view.

Work with the Capo-test is continuing by the Author but insufficient data is available to confirm the Manufacturers' claim that correlation with compressive strength is the same as for the Lok-test. It does appear, however, that the within-test scatter is not appreciably different despite the level of skill required to successfully perform the cutting operations.

4.3.2.3 Pull-off Method

This method has been developed in recent years at Queen's University Belfast and used successfully on both High Alumina and Portland Cement concretes in the field. The method is outlined in Paper 11 whilst Long (117) has recently provided more extensive data confirming the value and reliability of the approach which is likely to be more widely used in the future.

4.3.2.4 Break-off Method

This is described in Paper 11 and although commercially available from Norway has received little attention in the United Kingdom despite its potential value in circumstances where concrete tensile strength is important.

4.4 MATURITY MEASUREMENTS

Maturity is a function of the temperature history of the concrete with time, and it has been suggested by many workers that this may be calibrated against compressive strength for a particular concrete mix and curing circumstances. Naik (118) has recently reviewed the situation whilst Carino (119) has emphasised the influence of temperatures within the first six hours upon subsequent behaviour. Petersen (120) has also indicated the value of combined Lok-tests and maturity measurements based on a disposable chemically based device. It would seem doubtful that either this device (COMA-meter) or automatic electrically based integrating maturity meters which are available could accommodate this early age effect under extreme conditions. Maturity assessment based on within-pour temperature measurements is currently under investigation by the Author as part of a S.E.R.C. backed project into early age insitu strength assessment associated with formwork stripping times. This project is being executed with the collaboration of the C.E.G.B. and involves site measurements on cooling towers whilst under construction. In the course of the investigations the value of early age maturity/strength relationships is under examination, and comparisons are to be made between maturities based on temperature records and those indicated by COMA-meters for a range of early temperature regimes.

5. INTERPRETATION

The importance of discussion and agreement about the interpretation of insitu test results, prior to execution of testing, has been identified in Paper 16. As well as detailed planning of types, locations, and numbers of tests this need for agreement involves the acceptability of calibrations, likely accuracy of strength predictions, and the method of application of the results to specifications and design calculations.

Although individual technical papers have sometimes discussed the accuracy and applications of particular test methods, very little detailed guidance has been available to Engineers concerning planning and the interpretation of insitu test results until recent years. B.S. 6089 (102) published in 1981 provides the only 'official' guidance at present although B.S. 1881: Pt. 201, which has been referred to previously, will offer much more detailed guidance upon planning and test selection in due course.

5.1 PLANNING

The Author has on many occasions emphasised the importance of planning of any programme of insitu testing and the particular need to establish the aims of the investigation.

5.1.1 Selection of Test Methods

Figure 1 of Paper 6 provides a simple sequence for selection of strength testing procedures which has been enhanced in Figure 1 of Paper 17. Figure 1 of Paper 18 proposes a planning sequence encompasing all forms of insitu testing. Many factors to be considered at the planning stage are listed in Paper 18 whilst individual factors are examined in detail in Paper 16. Selection of strength tests may be assisted by Table 1.2 of Paper 16, which summarises the level of damage and principal restrictions associated with each method, and Table 1.3 which compares their relative merits. Table 1 of Paper 18 identifies important features of all principal non-destructive test methods, including those used for strength assessment. This table forms the basis of more extensive tables of similar nature in the Draft of B.S. 1881: Pt. 201, whilst much of the other material in Paper 18 is also included in that document.

5.1.2 Numbers and Locations of Tests

This is discussed in detail in section 1.3.3 of Paper 16 and Table 1.5 compares the relative numbers of readings necessary for various test methods.

5.2 CONCRETE STRENGTH VARIABILITY

The importance of variations of concrete quality within structural members was identified by Elvery (121) in 1969, but it is only more recently that the availability of increased data has led to a more general awareness of within-member variations and the differences between insitu strengths and those of standard specimens. These factors were considered in detail by the Author in Paper 19 in 1981 in relation to the planning and interpretation of insitu testing. Results obtained by the Author on 500mm deep reinforced concrete beams using three different test methods are presented in that Paper. These illustrate the presence of strength gradients and provide a numerical measure of variability in the form of coefficients of variation. Insitu strengths are compared with standard specimens, and consideration is also given to the significance of moisture condition and specimen size when considering cube strengths. These two latter effects being particularly important when developing calibrations for insitu testing.

In Paper 16 the Author has subsequently combined his own results with published data from a wide range of sources to summarise typical within-member variations according to member type in Figure 1.3, and insitu strength levels in Table 1.6 and Figure 1.6. Further data relating to slabs, showing 30% top to bottom strength differentials with an average value of 70% standard specimen strength, has since been presented by Mohamed (122) working under the Author's supervision. The influence of these factors upon interpretation of insitu results is discussed in

Paper 16 together with the value of examination of variability by both graphical and numerical methods. The application of insitu results to Specifications and Design Calculations also involves detailed consideration of material variability and is considered in Paper 16 with illustrative examples given in the Appendix to that Paper.

5.3 ACCURACY OF INSITU STRENGTH ESTIMATES

This will be governed by the variability of the particular test method involved, the number of tests made, and the reliability of correlations between measured value and concrete strength property required. These factors have all been considered in detail in papers dealing with individual test methods, and maximum likely accuracies for some of the more commonly used methods are summarised in Table 1.8 of Paper 16.

5.3.1 Calibrations

The importance of comparability of laboratory produced calibrations with conditions actually encountered on site is identified in Paper 16. Comparability of moisture conditions is of particular importance when interpreting results whilst specimen size effects may also be significant and must be clearly identified.

The ease of obtaining reliable calibrations will vary according to test method but in all cases these must be obtained experimentally, preferably for the mix concerned. Attempts have been made to develop theoretical analyses for some of the test methods and these have been valuable in explaining, and justifying, observed characteristics of the tests and results. However, the heterogeneous non-isotropic nature of insitu concrete coupled with the influence of operator technique on results of some methods effectively prohibits the development of worthwhile theoretical strength correlations. In other cases, such as Rebound Hammer measurements, the test is of a purely empirical nature with few attempts having been made to provide detailed theoretical explanation.

It is important however that consideration is given to the nature of the property being measured and its applicability to the information required. This may be of particular significance when testing concrete at very early ages since various properties of the hardened concrete may not develop at the same rates.

5.3.2 Test Combinations

Combinations of test methods may provide greater accuracy of insitu strength estimates as indicated in section 1.6 of Paper 16 whilst the use of more than one method to provide confirmatory evidence is illustrated in Table 1 of Paper 19 and has been mentioned previously. The use of cores or flexural tests to calibrate non-destructive methods is illustrated in Papers 5 and 6.

5.3.3 Member Strength

In Figure 6 of Paper 6 comparisons of estimated cube strengths in small section pretensioned x-beams obtained from cores and ultrasonic pulse velocities are made with values obtained from flexural tests. The relationship between the values from cores and those from flexural tests is less reliable than that earlier demonstrated by Cusens and Jackson (109), but this may be explained by the greater scatter associated with the smaller diameter specimens used by the Author, coupled with moisture effects. The closer agreement of the relationship between values based on ultrasonic pulse velocity and those from flexural tests confirms the value of ultrasonic pulse velocity when estimating member strength as earlier indicated by Elvery (121). Comparisons are also made in Figure 4 of Paper 17 between measured collapse moments and estimated cube strengths from small diameter cores for a series of T-section pre-tensioned purlins removed from a structure. In this case the value of core tests to predict member flexural capacity is clear, with the theoretical values based on material strength providing a lower bound to member capacity.

In Paper 20, the use of measured insitu strength values in conjunction with computer analyses of the behaviour of more complex structural components is examined in some detail. The Author's contribution relates principally to the sections dealing with assessment of insitu concrete strength. The case histories summarised in Paper 17 also illustrate a number of further practical factors that may be of significance when the principal aim of an investigation is estimation of the member or structure strength rather than the material strength alone. The influence of un-designed composite actions and construction deficiencies may be consider-able, and in-service deflection is concluded to be an unreliable guide to concrete strength deterioration.

6. CONCLUSIONS

Despite the developments within recent years the accuracy with which insitu concrete strength may be assessed is still limited. However, whilst precision may be inadequate for some purposes, there are very many situations in which careful selection, execution and interpretation of insitu testing may be extremely worthwhile. Civil and Structural Engineers are by nature cautious in their acceptance of new approaches and insitu testing has in some instances been discredited, usually because it has failed to satisfy unrealistic expectations. Developments have occurred in other countries which have been regarded with scepticism in the United Kingdom and in some instances this has been shown to be justified.

Fifteen years ago, at the time of the Conference held by the Institution of Civil Engineers in 1969 from which reference (121) is taken, not even Surface Hardness Measurements or Ultrasonic Pulse Velocity testing were included in British Standards. The present situation in which B.S. 6089 (102) exists and major revisions and extensions to B.S. 4408 are at an advanced stage reflects the advances that have been made and the increased interest by Engineers in insitu testing.

One of the most important aspects of reliable insitu strength assessment is that the investigating Engineer must be given the fullest possible information relating to material composition and construction procedures to permit reliable interpretation of results. It is also essential that all parties involved understand the significance of the values obtained.

Further work is clearly required in many areas to refine testing procedures, develop additional data, and to extend the applicability of insitu strength tests, especially for very early ages of the concrete. The other major area in which further information is required to assist the interpretation of insitu test results involves the gathering of insitu strength data relating to a much wider range of existing structures than is presently available. The practical use of insitu strength assessment will continue to increase as confidence in the validity of the results is established. It is hoped that the work by the Author has gone some way towards improving knowledge and understanding which may lead to more widespread acceptance of the benefits available.

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Paper 1

"Cores"

The Testing of Concrete in Structures

Surrey University Press 1982

Chapter 5 and Appendix B

5 Cores

The examination and compression testing of cores cut from hardened concrete is a well-established method, enabling visual inspection of the interior regions of a member to be coupled with strength estimation. Other physical properties which can be measured include density, water absorption, indirect tensile strength and movement characteristics, whilst cores are frequently used as samples for chemical analysis following strength testing. In most countries standards are available which recommend procedures for cutting, testing and interpretation of results; BS 1881 pt. 4 (24) in the UK whilst in the USA ASTM C.42 (65) and ACI 318–71 (66) are used. Extremely valuable and detailed supplementary information and guidance is also given by Concrete Society Technical Report No. 11 (4).

5.1 General procedures for core cutting and testing

5.1.1 *Core location and size*

Core location will be governed primarily by the basic purpose of the testing, bearing in mind the likely strength distributions within the member, discussed in Chapter 1, related to the expected stress distributions. Where serviceability assessment is the principal aim, tests should normally be taken at points where likely minimum strength and maximum stress coincide, for example from the top surface at near midspan for simple beams and slabs, or from any face near the top of lifts for columns or walls. If the member is slender, however, and core cutting may impair future performance, cores should be taken at the nearest non-critical locations. Aesthetic considerations concerning the appearance after coring may also sometimes influence the choice of locations. Alternatively, areas of suspect concrete may have been located by other methods.

If specification compliance determination is the principal aim, the cores should be located to avoid unrepresentative concrete, and for columns, walls or deep beams will normally be taken horizontally at least 300 mm below the

top of the lift. If it is necessary to drill vertically downwards, as in slabs, the core must be sufficiently long to pass through unrepresentative concrete which may occupy the top 20% of the thickness. In such cases drilling upwards from the soffit, if this is feasible, may considerably reduce the extent of drilling, but the operation may be more difficult. Reinforcing bars passing through a core will increase the uncertainty of strength testing, and should be avoided wherever possible. The use of a covermeter to locate reinforcement prior to cutting is therefore recommended.

Where the core is to be used for compression testing, a minimum diameter of 100 mm is required by both British and American standards although in Australia a 75 mm diameter is considered acceptable. In exceptional circumstances smaller diameters are used, especially in small-sized members where a 100 mm hole would be unacceptable, but the interpretation of results from small cores is more complex, and is considered separately in section 5.3. In general, the accuracy decreases as the ratio of size of aggregate to core diameter increases, and 150 mm is regarded as the preferred size for aggregates of up to 40 mm. 100 mm cores must not be used if the maximum aggregate size exceeds 25 mm, and this should preferably be less than 20 mm for 75 mm diameter cores. The choice of core diameter will also be influenced by the length of specimen which is possible. It is generally accepted that cores for compression testing should have a length/diameter ratio of between 1.0 and 2.0, but opinions vary concerning the optimum value.

The Concrete Society Technical Report No. 11 (4) suggests that cores should be kept as short as possible ($l/d = 1.0 \rightarrow 1.2$) for reasons of drilling costs, damage, variability along length, and geometric influences on testing. Whilst these points are valid, procedures for relating core strength to cylinder or cube strength usually involve correction to an equivalent standard cylinder with $l/d = 2.0$, and it can be argued that uncertainties of correction factors are minimized if the core length/diameter ratio is close to 2.0 (see section 5.2.2). BS 1881 (24) gives correction factors over the whole range, but these have been shown to be of doubtful accuracy, and BS 6089 (10) supports the Concrete Society (4) view.

The number of cores required will depend upon the reasons for testing and the volume of concrete involved. The likely accuracies of estimated strength are discussed in section 5.2.3, but the number of cores must be sufficient to be representative of the concrete under examination as well as provide a strength estimate of acceptable accuracy.

5.1.2 *Drilling*

A core is usually cut by means of a rotary cutting tool with diamond bits, as shown in Figure 5.1. The equipment is portable, but it is heavy and must be

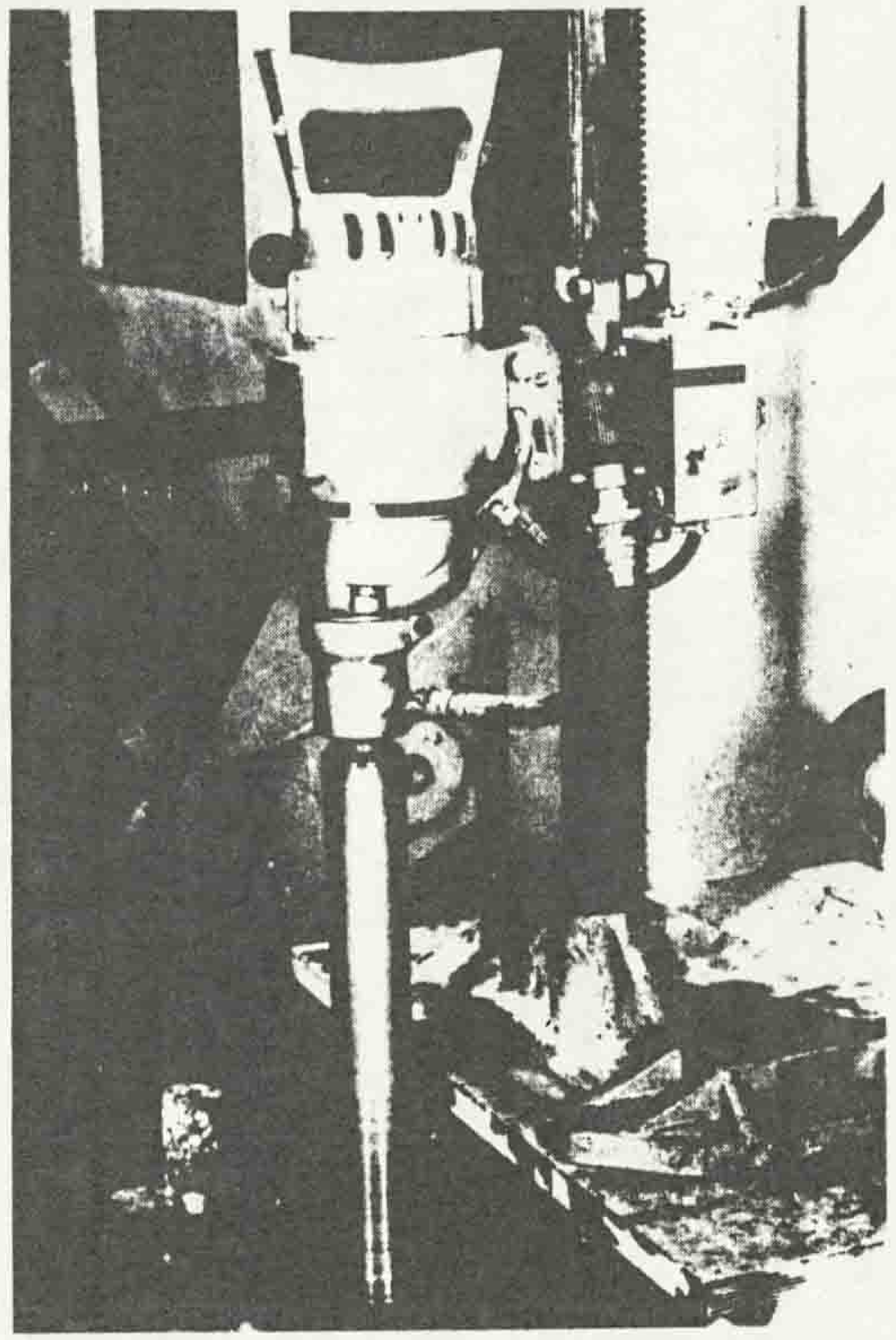

Figure 5.1 Core cutting drill.

firmly supported and braced against the concrete to prevent relative movement which will result in a distorted or broken core, and a water supply is also necessary to lubricate the cutter. Uniformity of pressure is important, so it is essential that drilling is performed by a skilled operator. A cylindrical specimen is obtained, which may contain embedded reinforcement, and which will usually be removed by breaking off by insertion of a cold chisel down the side of the core, once a sufficient depth has been drilled. The core, which will have a rough inner end, may then be removed using the drill or tongs, and the hole made good. This is best achieved either by ramming a dry, low shrinkage concrete into the hole, or by wedging a cast cylinder of suitable size into the hole with cement grout or epoxy resin. It is important that each core is examined at this stage, since if there is insufficient length for testing, or excessive reinforcement or voids, extra cores must be drilled from adjacent locations. Each core must be clearly labelled for identification, with the drilling surface shown, and cross-referenced to a simple sketch of the element drilled. Photographs of cores are valuable for future reference, especially as

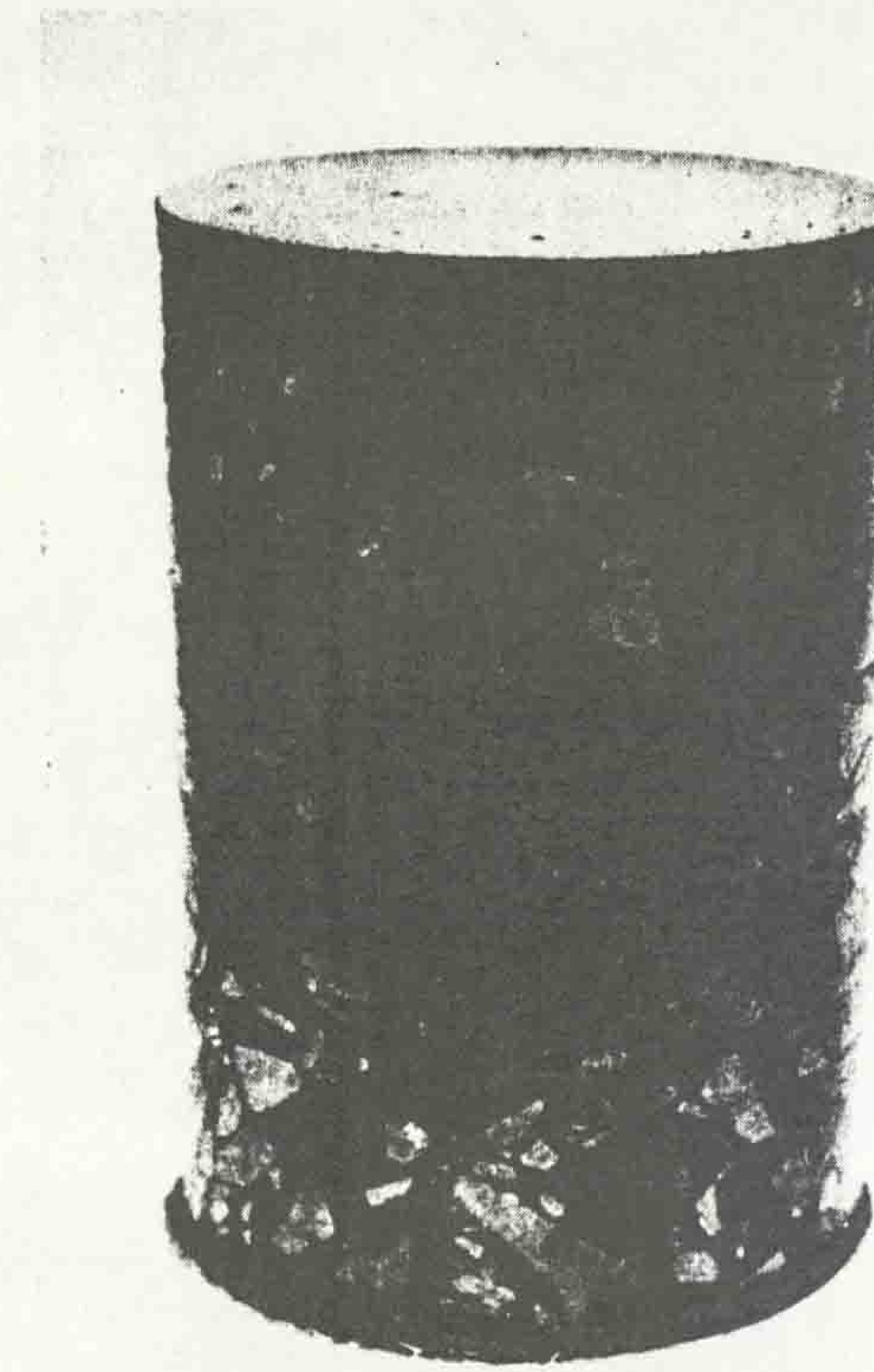

Figure 5.2 Typical core.

confirmation of features noted during visual inspection, and these should be taken as soon as possible after cutting. A typical photograph of this type is shown in Figure 5.2.

5.1.3 *Testing*

Each core must be trimmed and capped before visual examination, assessment of voidage, and density determinations.

5.1.3.1 Visual examination. Aggregate type, size and characteristics should be assessed together with grading. These are usually most easily seen on a wet surface, but for other features to be noted, such as aggregate distribution, honeycombing, cracks, defects and drilling damage, a dry surface is preferable. Precise details of the location and size of reinforcement passing through the core must also be recorded. The voids should be classified, either in general terms on the basis of size and numbers as proposed by BS 1881 pt. 4 (24) or, if a quantitative value of excess voidage is required for use in calculating the potential strength (see section 5.2.2), this may be assessed by comparison with "standard" photographs of known voidage. Concrete Society Technical Report 11 (4) contains such reference photographs with excess voidages based on a fully compacted "potential" voidage of 0.5 %. The BS 1881 (24) classification method is based on small voids (0.5–3 mm), medium voids (3–6 mm), and large voids (> 6 mm), and these are counted and expressed in terms of the number of each group per 100 000 mm^2 of cut face. This is roughly equivalent to a 150 mm core of $l/d = 1.5$. The extent of voids is then reported on the basis of the values contained in Table 5.1.

Table 5.1 Classification of extent of voids (ref. 24)

	No. of voids/100 000 mm² of cut face		
	Small	*Medium*	*Large*
Negligible	< 40	< 4	0
Few	40–150	4–15	< 2
Considerable	150–400	15–75	2–15
Numerous	>400	>75	>15

5.1.3.2 Trimming. Trimming, preferably with a masonry or diamond saw, should give a core of a suitable length with parallel ends which are normal to the axis of the core. If possible, reinforcement and unrepresentative concrete should be removed.

5.1.3.3 Capping. Cores should be capped with high alumina cement mortar or sulphur-sand mixture to provide parallel end surfaces normal to the axis of the core. (Other materials should not be used as they have been shown to give unreliable results.) Caps should be kept as thin as possible, but if the core is hand trimmed they may be up to about the maximum aggregate size at the thickest points.

5.1.3.4 Density determination. This is recommended in all cases, and is best measured by the following procedure (4):

(a) Measure volume (V_u) of trimmed core by water displacement.
(b) Establish density of capping material (D_c).
(c) Before compressive testing, weigh soaked/surface dry capped core in air and water to determine gross weight W_t and volume V_t.
(d) If reinforcement is present this should be removed from the concrete after compression testing, and the weight W_s and volume V_s determined.
(e) Calculate saturated density of concrete in the uncapped core from

$$D_a = \frac{W_t - D_c(V_t - V_u) - W_s}{V_u - V_s}.$$

If no steel is present W_s and V_s are both zero.

The value thus obtained may be used, if required, to assess the excess voidage of the concrete using the relationship

$$\text{estimated excess voidage} = \frac{D_p - D_a}{D_p - 500} \times 100\,\%$$

where D_p = the potential density based on available values for 28 day old cubes of the same mix.

5.1.3.5 Compression testing. The standard procedure in the United Kingdom is to test cores in a saturated condition, although in some countries dry testing is used if the in-situ concrete is in a dry state. If the core is to be saturated, testing should be not less than 2 days after capping and immersion in water. The mean diameter must be measured to the nearest 2 mm by caliper, with measurements on two axes at $\frac{1}{4}$ and mid-points along the length of the core, and the core length also measured to the nearest 5 mm.

Compression testing will be carried out at a rate of 15 N/mm^2/min in a suitable testing machine and the mode of failure noted. If there is cracking of the caps, or separation of cap and core, the result should be considered as being of doubtful accuracy. Ideally cracking should be similar all round the circumference of the core, but a diagonal shear crack is considered satisfactory, except in short cores or where reinforcement or honeycombing is present.

5.1.3.6 Other tests on cores. Whilst compression testing as described above is by far the most common method of testing cores for strength, recent research has indicated the potential of two other methods which are outlined below. Both of these measure the tensile strength, although neither method is yet fully established. Tests for other properties of the concrete, such as permeability or air content (Chapters 7 and 8) may also be performed on suitably prepared specimens obtained from cores.

In the *Point Load Strength Test*, Robins (67) has shown that the point load test, which is an accepted method of rock strength classification, may usefully be applied to concrete cores. A compressive load is applied across the diameter (Figure 5.3) by means of a manually operated hydraulic jack, with

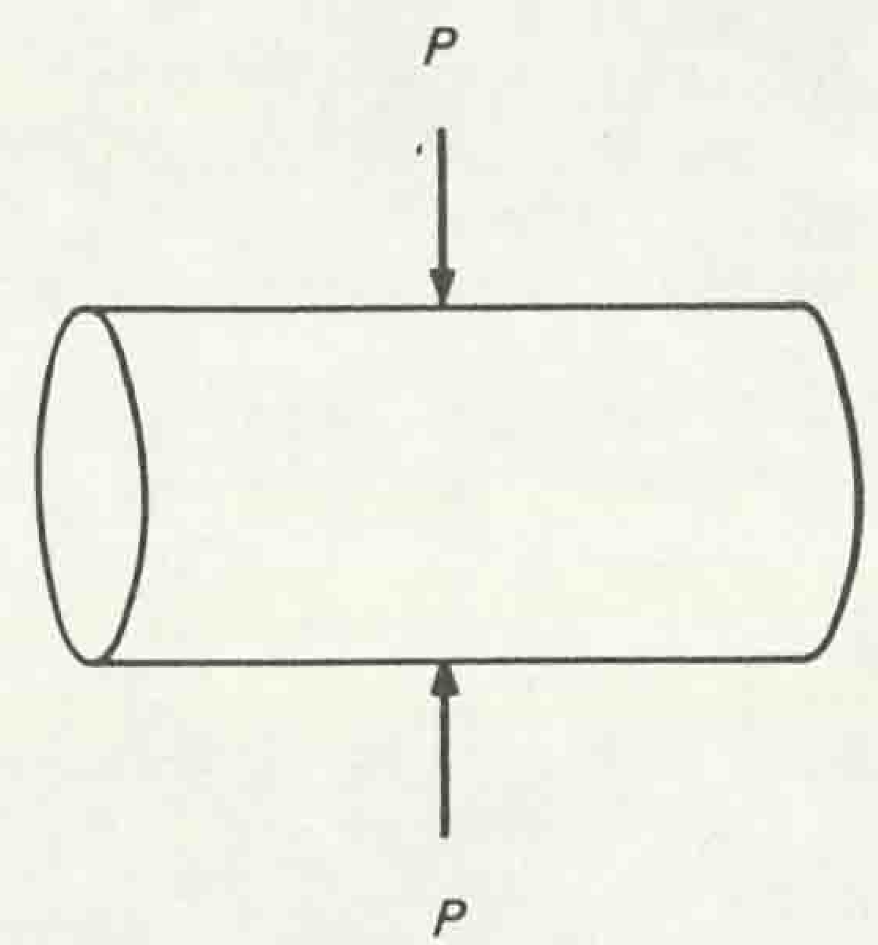

Figure 5.3 Point load test.

the specimen held between spherically truncated conical platens with a point of 5 mm radius. It has been found that the point load strength index is indirectly related to the concrete compressive strength, although core size and aggregate type affect the relationship. For a given aggregate and core size, the index varies linearly with cube strength for strengths greater than 20 N/mm^2. Robins (67) also claims that the testing variability is comparable to that expected for conventional core testing. The advantages of this approach are that trimming and capping are not required and that the testing forces are lower, thus permitting the use of small portable equipment on site at a reduced unit cost.

The point load test is essentially a tensile test, but data relating results to other forms of tensile testing are unfortunately not available. The greatest potential for future development of this method may perhaps lie as a means of estimating in-situ tensile strength.

In the *Gas Pressure Tension Test*, Clayton (68) has demonstrated that applied gas pressure may be used on cylinders to simulate the effects of uniaxial tensile tests, and that cores may also be used for this purpose. The specimen is inserted into a cylindrical steel jacket with seals at each end, and gas pressure is applied to the bare curved surface. Nitrogen has been found to be safe and convenient. The flow is controlled by a single-stage regulator, and a pressure gauge is used for measurement. Pressure is increased manually at a specified rate, until failure occurs by the formation of a single cleavage plane transverse to the axis of the specimen. The two sections are forced violently apart and safety precautions are necessary to prevent ejection of the fragments from the testing jacket.

The method has been developed using 100 mm cylinders, but has been successfully applied to 75 mm cores of high alumina cement concrete which in some cases had length/diameter ratios of less than 1.0. Whilst preliminary

evidence suggests that this may provide a reliable method of determining in-situ tensile strength, further research is necessary before such results can be regarded with confidence.

5.2 Interpretation of results

5.2.1 *Factors influencing measured core compressive strength*

These may be divided into two basic categories according to whether they are related to concrete characteristics or testing variables.

5.2.1.1 Concrete characteristics. The moisture condition of the core will influence the measured strength—a saturated specimen has a value 10–15% lower than a comparable dry specimen. It is thus very important that the relative moisture conditions of core and in-situ concrete are taken into account in determining actual in-situ concrete strengths. If the core is tested whilst saturated, however, comparison with standard control specimens which are also tested saturated, will be straightforward.

The curing regime, and hence strength development, of a core and of the parent concrete will be different from the time of cutting. This effect is very difficult to assess, and in mature concrete may be ignored, but for concrete of less than 28 days should be considered.

Voids in the core will reduce the measured strength, and this effect can be allowed for by measurement of the excess voidage when comparing core results with standard control specimens from the point of view of material specification compliance. Figure 5.4, based on reference (4), shows the

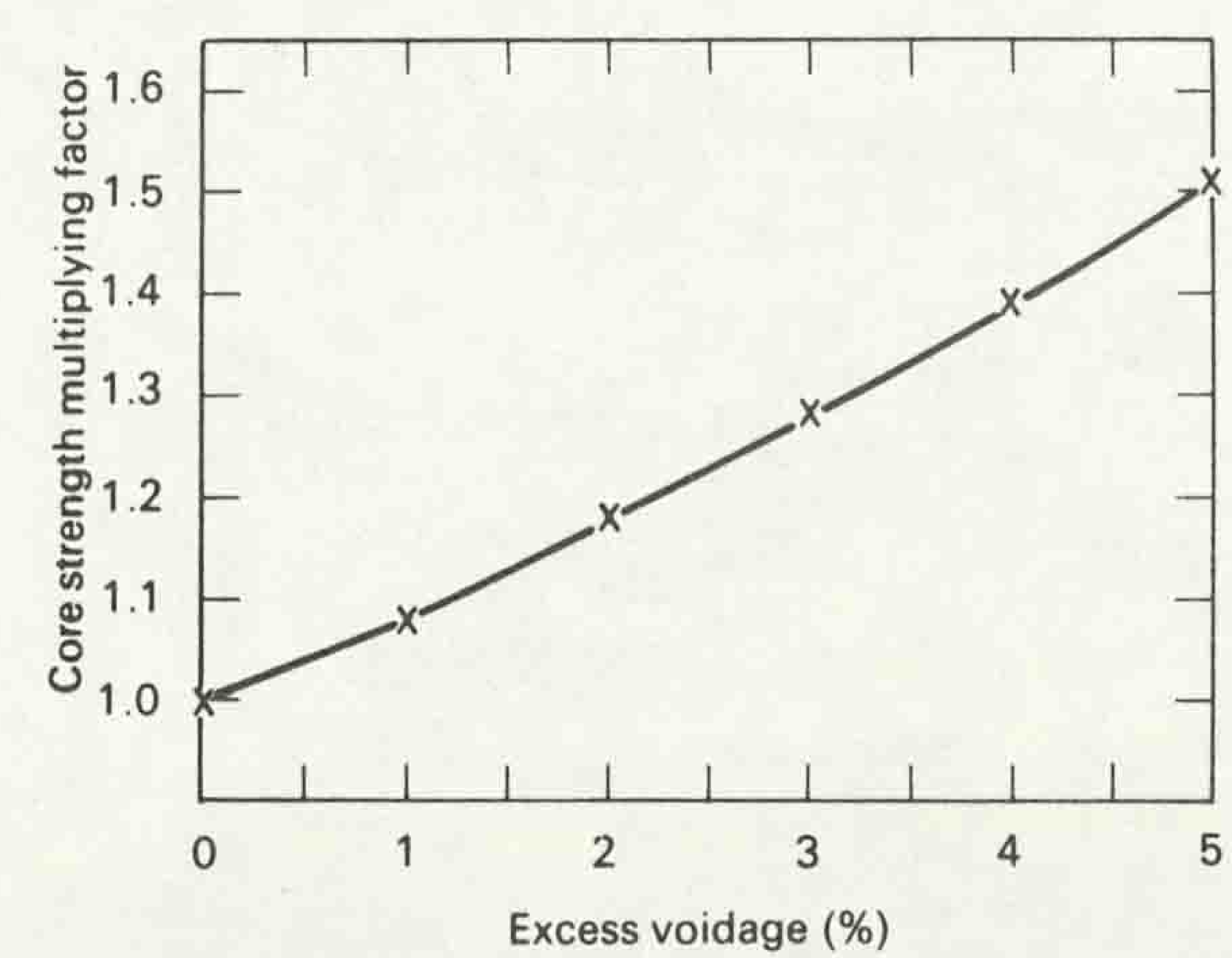

Figure 5.4 Excess voidage corrections (based on ref. 4).

influence of this effect. Under normal circumstances an excess voidage of less than 2.5% would be expected.

5.2.1.2 Testing variables. These are numerous, and in many cases will have a significant influence upon measured strength. The most significant factors are outlined below.

(a) Length/diameter ratio of core
As the ratio increases, the measured strength will decrease due to the effect of specimen shape on stress distributions whilst under test. Since the standard cylinder used in many parts of the world has a length/diameter ratio of 2.0, this is normally regarded as the datum for computation of results, and the relationship between this and a standard cube is established. Although there are discrepancies of published data on length/diameter effects, the values shown in Figure 5.5, based on the Concrete Society recommendations (4), have come to be regarded as the most realistic in recent years.

(b) Diameter of core
The diameter of core may influence the measured strength and variability (see section 5.1.1). Measured concrete strength will generally decrease as the specimen size increases; for sizes above 100 mm this effect will be small, but for smaller sizes this effect may become significant. As the diameter decreases, the ratio of cut surface area to volume increases, and hence the possibility of strength reduction due to cutting damage will increase. It is generally accepted that a minimum diameter/maximum aggregate ratio of 3 is required to make test variability acceptable, although a higher value (see section 5.1.1) should preferably be used.

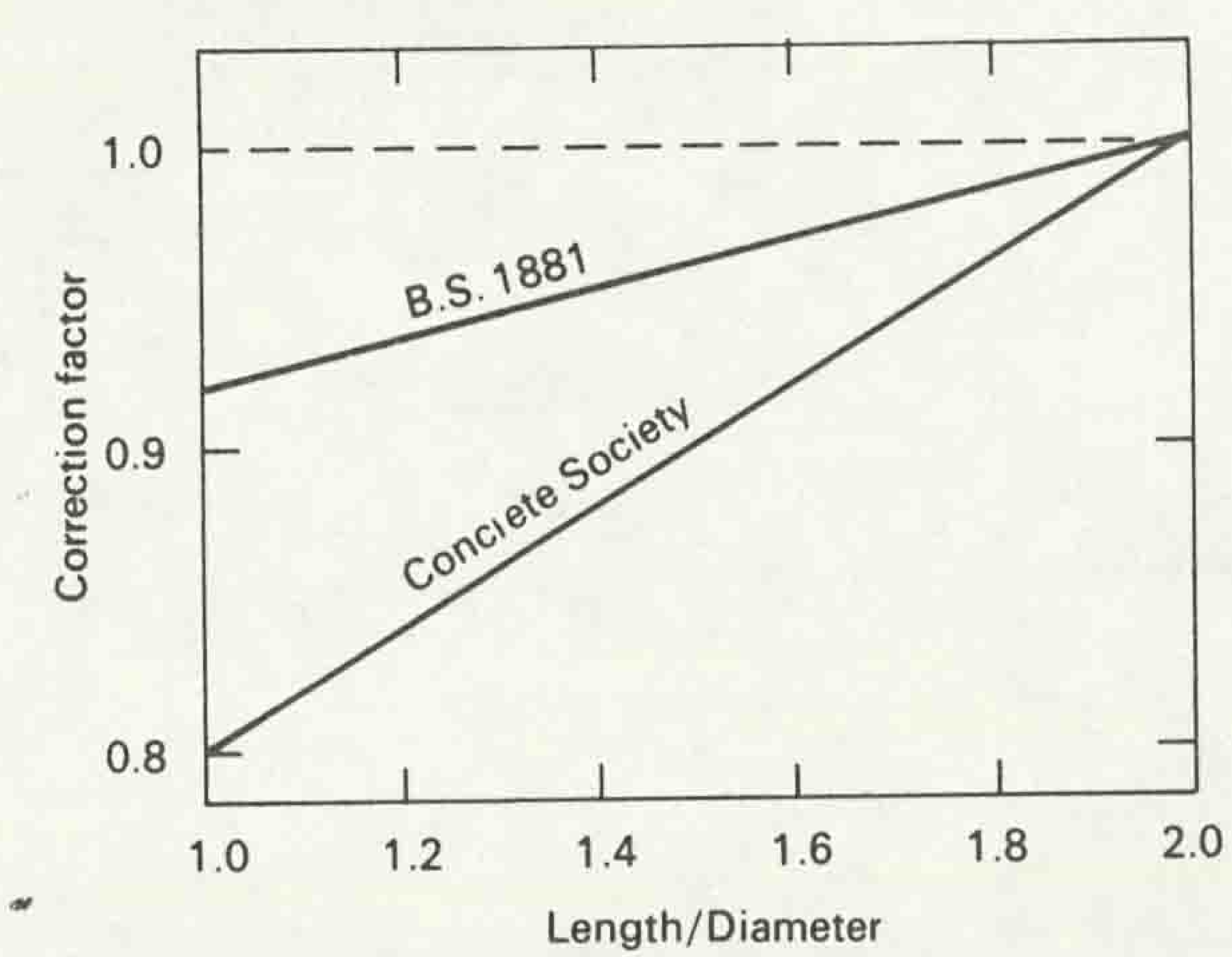

Figure 5.5 Length/diameter ratio influence (based on refs 4 and 24).

(c) Direction of drilling
As a result of layering effects, the measured strength of a specimen drilled vertically relative to the direction of casting is likely to be greater than that for a horizontally drilled specimen from the same concrete. Published data on this effect is variable, but an average difference of 8% is suggested (4). Whilst standard cylinders are tested vertically, cubes will normally be tested at right angles to the plane of casting and hence can be related directly to horizontally drilled cores.

(d) Method of capping
Provided that the materials recommended in 5.1.3.3 have been used, their strength is greater than that of the core, and the caps are sound, flat, perpendicular to the axis of the core and not excessively thick, the influence of capping will be of no practical significance.

(e) Reinforcement
Published research results indicate that the reduction in measured strength due to reinforcement may be less than 10%, but the variables of size, location and bond make it virtually impossible to allow for. Reinforcement must therefore be avoided whenever possible, but in cases where it is present the measured core strength must be corrected and treated with caution. It has been suggested (4) that for a core containing a bar perpendicular to the axis of the core the following correction factor may be applied to the measured core strength

$$\text{corrected strength} = \text{measured strength} \times \left[1.0 + 1.5\left(\frac{\phi_r}{\phi_c} \cdot \frac{h}{l}\right)\right]$$

where ϕ_r = bar diameter
ϕ_c = core diameter
h = distance of bar axis from nearer end of core
l = core length.

Multiple bars within a core can similarly be allowed for by the expression

$$\text{corrected strength} = \text{measured strength} \times \left[1.0 + 1.5\frac{\Sigma(\phi_r \cdot h)}{\phi_c \cdot l}\right].$$

If the spacing of two bars is less than the diameter of the larger bar, only the bar with the higher value of $(\phi_r \cdot h)$ should be considered.

5.2.2 *Estimation of cube strength*

Estimation of an equivalent cube strength corresponding to a particular core result must initially account for two main factors. These are

(a) the effect of the length/diameter ratio, which requires a correction factor, obtained from Figure 5.5, to be applied to convert the core strength to an equivalent standard cylinder strength, and
(b) Conversion to an equivalent cube strength using the generally accepted average relationship (4) cylinder strength = 0.8 × cube strength.

BS 1881 pt. 4 (24) recommends that equivalent cube strength estimates are obtained in this way by calculating a corrected core strength using the appropriate factor from Figure 5.5, followed by correction to an equivalent cube strength by multiplying by 1.25. The result should be quoted to the nearest 0.5 N/mm^2. The comments in section 5.2.1.2 concerning the validity of these particular length/diameter factors should be borne in mind; also, this approach takes no account of features such as orientation of drilling relative to casting. The results obtained are thus of doubtful value since they are unlikely to represent a realistic estimate of in-situ cube strength.

The Concrete Society Report (4) recommends a more detailed procedure which is based on the alternative correction factors of Figure 5.5, coupled with an allowance of 6% strength differential between a core with a cut surface relative to a cast cylinder. A strength reduction of 15% is also incorporated to allow for the weaker top surface zone of a corresponding cast cylinder, before conversion to an equivalent cube strength by the multiplication factor of 1.25. An 8% difference between vertical and horizontally drilled cores is also incorporated with the resulting expressions emerging:

Horizontally drilled core:

$$\text{estimated cube strength} = \frac{2.5 f_\lambda}{1.5 + 1/\lambda}.$$

Vertically drilled core:

$$\text{estimated cube strength} = \frac{2.3 f_\lambda}{1.5 + 1/\lambda}$$

where f_λ is the measured strength of a core with length/diameter = λ.

It is interesting to note that, using these expressions, the strength of a horizontally drilled core of length/diameter (λ) = 1 will be the same as the equivalent cube strength. The cube strengths evaluated in this way will be estimates of the actual in-situ strength of the concrete in a wet condition.

If equivalent cylinder strengths are required then ASTM C42 (65) recommends length/diameter correction factors similar to BS 1881 (24). ACI 318–71 (65) also suggests that an average in-situ strength of at least 85% the minimum specified value is adequate, and that cores may be tested after air-drying for 7 days if the structure is to be dry.

The strength differences between in-situ concrete and standard specimens have been fully discussed in Chapter 1. An average recommended relationship is that the "potential" strength of a standard specimen made from a particular mix is about 30% higher than the actual "fully compacted" in-situ strength (4). If this value is used to estimate a potential strength for comparison with specifications, the uncertainty of the relationship must be remembered. Appendix 3 of the Concrete Society Report (4) offers detailed guidance relating to curing history.

The expressions for cube strength will then become

Horizontally drilled core:

$$\text{estimated potential cube strength} = \frac{3.25 f_\lambda}{1.5 + 1/\lambda}.$$

Vertically drilled core:

$$\text{estimated potential cube strength} = \frac{3.0 f_\lambda}{1.5 + 1/\lambda}.$$

A worked example of evaluation of core results using the Concrete Society recommendations is given in Appendix B.

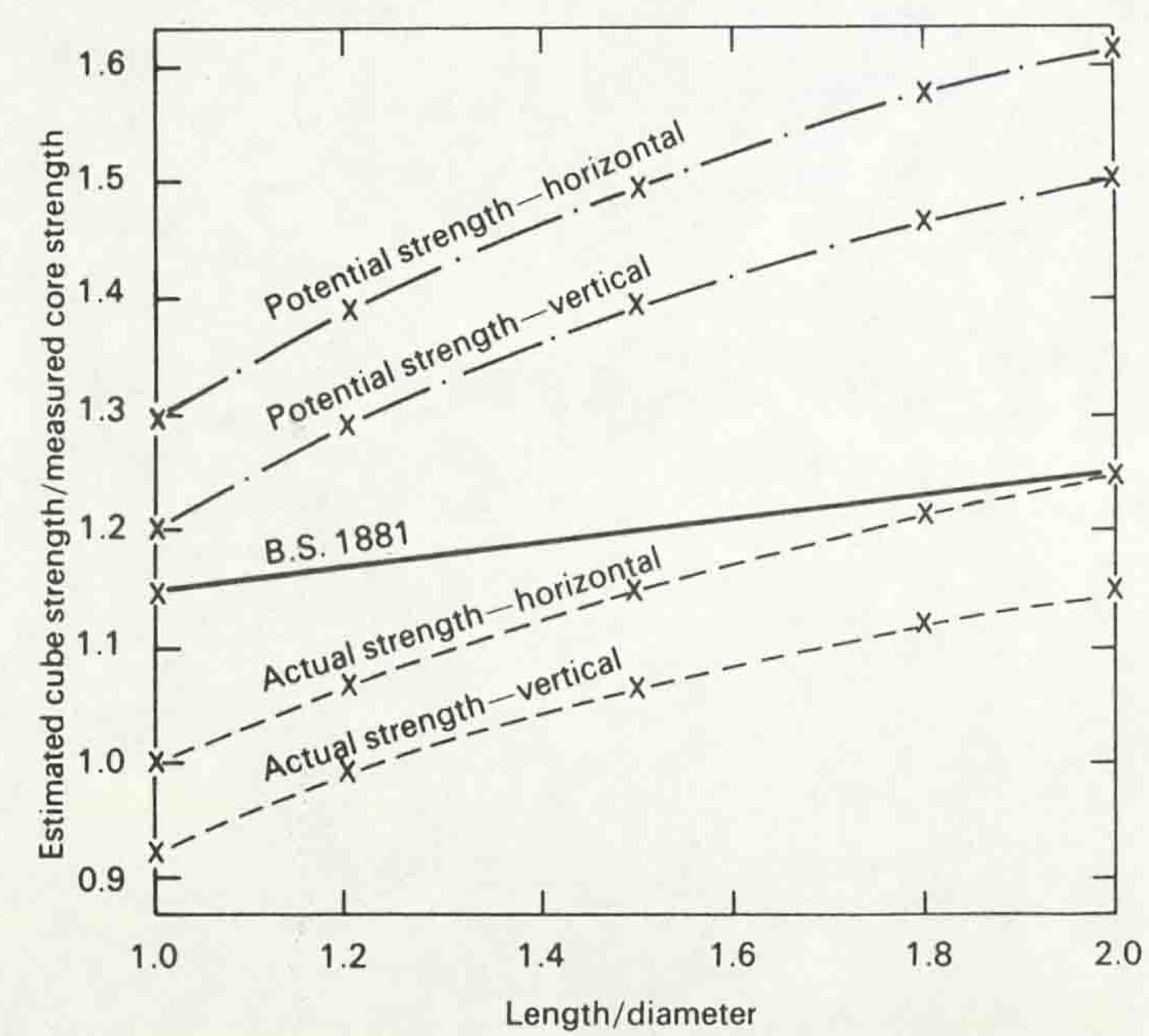

Figure 5.6 Effect of calculation method (based on refs 4 and 24).

The effect of the calculation method can be considerable, as illustrated in Figure 5.6, and this emphasizes the importance of agreement between all parties of the method to be used in advance of the testing.

5.2.3 *Reliability, limitations and applications*

The likely coefficient of variation due to testing effects is about 6% for carefully cut and tested cores, which can be compared with a corresponding value of 3% for cubes. This difference is largely caused by the effects of cutting, especially since cut aggregate particles are only partially embedded in the core and may not make a full contribution during testing. It is claimed that the likely 95% confidence limits on actual strength prediction for a single core is ±12% when the Concrete Society calculation procedures (4) are adopted. It follows that for a group of *n* cores, the 95% confidence limits on estimated actual in-situ strength are $\pm 12/\sqrt{n}\%$. Where the "potential" strength of the concrete is to be assessed, a minimum of four cores are required and an accuracy of better than ±15% cannot be expected. This can only be achieved if great care is taken to ensure that the concrete tested is representative, by careful location and preparation of the specimens.

Examination of Figure 5.6 shows disturbing differences between the results computed by the two methods currently in use in the UK. If the validity of the Concrete Society recommendations (4) are accepted, it will be clear that results computed by the BS 1881 (24) method are liable to overestimate the actual strength by 25%. The Concrete Society method makes detailed allowance for the many variable factors influencing core results, and will provide the more reliable estimates of equivalent cube strengths.

Damage caused by drilling may be particularly significant for old brittle concretes, where internal cracking of the core may be aggravated by the loss of the confining effect of the surrounding body of concrete. The estimated cube strengths obtained from core compression tests may tend to underestimate the true in-situ capacity in this situation. Strength changes with age may also be considered when interpreting core results, but any allowances must be carefully considered as discussed in section 1.4.2.

The principal limitations of core testing are those of cost, inconvenience and damage, and the localized nature of the results. It is strongly recommended that core testing is used in conjunction with some other form of testing which is less tedious and destructive. The aim of this is to provide data on relative strengths within the body of the concrete under test. The size of core needed for reliable strength testing can pose a serious practical problem; "small" cores may be worthy of consideration with slender members. It may also be appropriate to consider using a larger number of "small" diameter cores to obtain an improved spread of test locations where large volumes of

concrete are involved. The cutting effort for three 50 mm cores may be as low as one-third of that for one 150 mm specimen, and a comparable overall strength accuracy may be expected (see section 5.3.2) provided that maximum aggregate size is less than 17 mm. Where cores are used for other purposes, it will often be possible to use a "small" diameter with considerable savings of cost, inconvenience and damage.

Apart from physical testing, cores often provide the simplest method of obtaining a sample of the in-situ concrete for a variety of purposes, but care must be taken that the effects of drilling, including heat generated by friction, or the presence of water, do not distort the subsequent results. A sample taken from the centre of a core may conveniently overcome this problem. Chemical analysis can often be performed on the remains of a crushed core, or specimens may be taken specifically for that purpose. Visual inspection of the interior of the concrete may be extremely valuable both for the assessment of compaction and workmanship, and for obtaining basic data about concrete for which no records are available. In cases where structural assessments of old structures are required, cores may also prove valuable in confirming covermeter results concerning the location and size of reinforcement.

5.3 Small cores

Whilst standards normally require cores to have a minimum diameter of 100 mm for compressive strength testing, cores of smaller diameter offer considerable advantages in terms of reduced cutting effort, time and damage. For applications such as visual inspection, density, or voidage determination, reinforcement location or chemical testing, these savings may be valuable. However, the reliability of small diameter cores for compression testing is lower than for "normal" specimens. The many factors which affect normal core results may be also expected to influence small cores, but the extent of these factors may vary and other effects which are normally unimportant may become significant.

5.3.1 *Influence of specimen size*

It is well established that measured concrete strength usually increases as the size of the test specimen decreases, and that results tend to be more variable. This latter effect has been confirmed for core specimens by Henzel and Freitag (69). The ratio of cut surface area to volume increases as diameter decreases and hence the potential influence of drilling damage is increased. Also, the ratio of aggregate size to core diameter is increased and may possibly exceed the generally recognized acceptable limit of 1:3. Kesler (70) has shown that concrete strength is a further factor that may influence the behaviour of a core. These various factors are inter-related and difficult to isolate. For example increased strength due to small specimen size may be offset by a reduction due to cutting effects.

The most common diameter for small cores is 40–50 mm. The author has reported extensive laboratory tests to investigate the behaviour of 44 mm specimens (71), in which a total of 23 mixes were used, ranging from 10 to 82 N/mm^2 with 10 and 20 mm gravel aggregates, and cores were cut from 500 × 100 × 100 mm laboratory cast prism specimens to provide a range of length/diameter ratios.

5.3.1.1 Length/diameter ratio. The average relationship for length/diameter effects on 44 mm cores (71) is shown in Figure 5.7 which compares the relationships discussed in section 5.2.1.2 for normal cores. This relationship

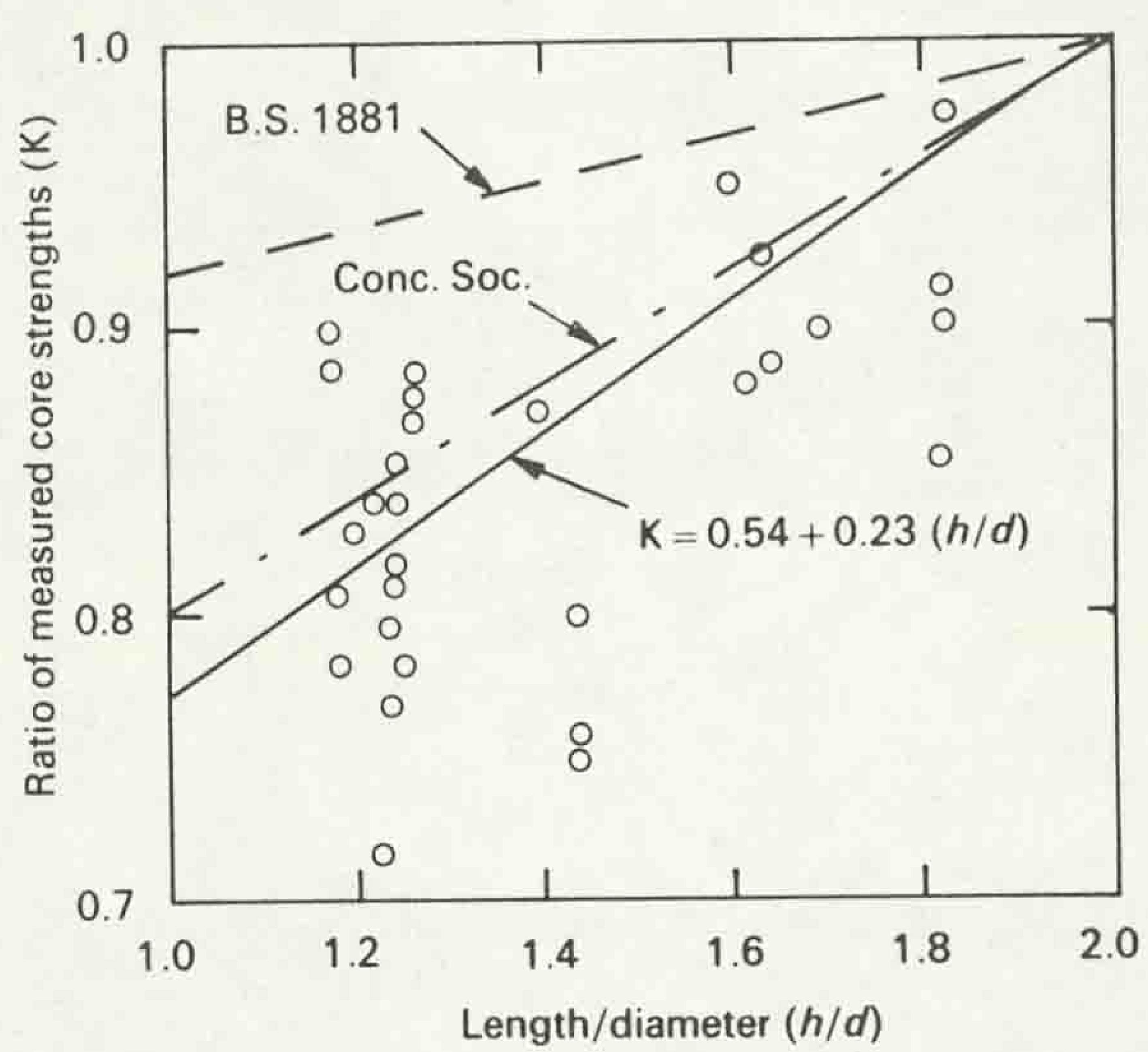

Figure 5.7 Length/diameter ratio for small cores (based on refs 4, 24 and 71).

was found to be independent of drilling orientation, aggregate size and cement type, for practical purposes, although the scatter of results is high since each point in Figure 5.7 represents the average of four similar cores. It will be seen that the correction factor for length/diameter ratio is close to the Concrete Society recommendation (4) for larger cores.

5.3.1.2 Variability of results. No significant change of variability was found between the extremes of length/diameter ratio for either aggregate size, and the average coefficient of variation of 8 % was also independent of orientation.

However, taking account of concrete variability as indicated by control cubes, it is clear that 20 mm aggregate cores show a higher variability due to cutting and testing than 10 mm aggregates (71). The range of coefficients of variation of strength for groups of similar cores was large, and made identification of the effects of other variables impossible to assess. Bowman (72) has reported a coefficient of variation of 28.9 % for 50 mm cores from in-situ concrete on a site in Hong Kong compared with a value of 19.5 % for corresponding 150 mm cores from the same concrete.

5.3.1.3 Measured strength. Based on the author's tests (71) the factors required to convert the measured core strength (after correction to $l/d = 2.0$) to an equivalent 100 mm cube strength are given in Table 5.2. If an equivalent 150 mm cube strength is required, these values may be reduced by 4 %.

Table 5.2 Cube/corrected core conversion factors for 44 mm cores with $l/d = 2.0$ (ref. 71)

Core orientation		Maximum aggregate size		
		10 mm	*20 mm*	*Combined*
Vertical	Conversion factor to 100 mm cube	1.05	1.25	1.15
	95 % confidence limit on predicted cube strength (4 cores)	±17 %	±23 %	±23 %
Horizontal	Conversion factor to 100 mm cube	1.14	1.22	1.17
	95 % confidence limit on predicted cube strength (4 cores)	±15 %	±17 %	±17 %

It can be seen that for 10 mm aggregates, the vertically drilled cores are approximately 8 % stronger than comparable horizontally drilled specimens relative to cubes. This is as anticipated for larger specimens, but the measured strengths are approximately 10 % stronger than expected from the Concrete Society recommendations (4), resulting in a lower correction factor to obtain an equivalent cube strength.

With 20 mm maximum aggregate, however, the cores were considerably weaker relative to cubes, confirming the influence of the aggregate size/core diameter ratio discussed above. In this case the orientation effect could not be detected. It is suggested that 10 mm and 20 mm aggregate concrete should be treated separately when converting 44 mm cores to equivalent cube strength.

If this is done, the 95% confidence limits on the average of the results of groups of four cores of this size under laboratory conditions are unlikely to be better than the values given in table 5.2. These may be approximated by $\pm 36/\sqrt{n}\%$ when n is the number of cores in the group. Bowman's reported results (72) also show a 7% higher strength for 50 mm cores when compared with 150 mm cores, but the aggregate size is not indicated.

5.3.2 *Reliability, limitations and applications*

The reliability of compressive tests on small diameter cores is known to be less than for "normal" specimens, and the author has suggested a factor of 3 × applied to the 95% confidence limits of predicted actual cube strengths under laboratory conditions. This gives a value of $\pm 36/\sqrt{n}\%$ for n cores with an aggregate size/diameter ratio of less than 1:3. But if the ratio of aggregate size/diameter is greater than 1:3, this accuracy is likely to decrease and may be as low as $\pm 50/\sqrt{n}\%$. Site cutting difficulties may further reduce accuracy. All the procedures described in section 5.1 concerning location, drilling and testing must be followed, just as for larger cores, and the effects of excess voidage, moisture and reinforcement accounted for as described in section 5.2. Particular care must be taken to ensure that the core is representative of the mass of the concrete, and this is particularly critical in slabs drilled from the top surface in view of the reduced drilling depth required for a small core.

There is no doubt that for applications other than compressive strength testing, small cores offer many economical and practical advantages compared with larger specimens. These applications include visual examination (including materials and mix details, compaction, reinforcement location and sizing); density determination; other physical tests, including point load or gas pressure tests; and chemical testing. For compressive strength testing, the chief limitation is variability of results and consequent lack of accuracy of strength prediction, unless many more specimens are taken than would normally be necessary. At least three times the number of "standard" cores is required to give comparable accuracy, but it can be argued that this would still require less drilling in many instances and permits a wider spread of sample location. It is clear that considerable differences of predicted cube strength will arise from use of the various calculation methods, and as for larger cores it is essential that agreement is reached between all parties, before testing, about the method to be used.

Bowman (72) has described a successful approach in which 50 mm cores were used for strength tests on cast in-place piles because of their cheapness and ease of cutting, but were backed up by 150 mm cores where results were on the borderline of the specification. Another common situation in which small cores may be necessary for strength testing is when the slenderness of

the member does not permit a larger diameter from the point of view of continued serviceability or adequate length/diameter ratio (> 1.0). This will apply especially to prestressed concrete members. Whilst in such cases small diameters are inevitable it is essential that the engineer fully appreciates the limitations of accuracy that he may expect. It may be that some other non-destructive approach will yield comparable accuracies of strength prediction, according to the availability of calibrations, with less expense, time and damage.

Appendix B: Example of evaluation of core results

A 100 mm diameter core drilled horizontally from a wall of concrete with 20 mm maximum aggregate size contains one no. 20 mm reinforcing bar normal to the core axis and located at 35 mm from one end. Measured water-soaked concrete density = 2320 kg/m^3 after correction for included reinforcement (section 5.1.3.4).
Measured crushing force = 160 kN (following BS 1881 (24) procedure).
Failure mode—normal.
Measured core length after capping = 120 mm.

(*a*) *BS1881* (*24*) *Equivalent cube strength*

$$\text{Measured core strength} = \frac{160 \times 10^3}{\pi \times \frac{100^2}{4}} = 20.5\ \mathrm{N/mm^2}.$$

Core length/diameter ratio = 120/100 = 1.2.
Length correction factor = 0.93 (Figure 5.5).
Corrected cylinder strength = $20.5 \times 0.93 = 19.0\ \mathrm{N/mm^2}$.
Equivalent cube strength = $19.0 \times 1.25 = 24.0\ \mathrm{N/mm^2}$.

(*b*) *Concrete Society* (*4*) *Actual cube strength*

$$\text{Measured core strength} = \frac{160 \times 10^3}{\pi \times \frac{100^2}{4}} = 20.5\ \mathrm{N/mm^2}.$$

Core length/diameter ratio = 120/100 = 1.2.

$$\text{Estimated actual cube strength} = \frac{2.5}{\left(1.5+\frac{1}{1.2}\right)} \times 20.5 \quad \text{for a horizontal core.}$$

$$= 22\ \text{N/mm}^2$$

$$\text{Reinforcement correction factor} = 1 + 1.5\left(\frac{20}{100} \times \frac{35}{120}\right)$$

$$= 1.09.$$

Corrected actual cube strength $= 22 \times 1.09\ \text{N/mm}^2 \pm 12\%$ for an individual result.

$$= 24.0 \pm 3\ \text{N/mm}^2$$

(*c*) *Concrete Society* (*4*) *Potential cube strength*

Measured core strength $= 20.5\ \text{N/mm}^2$.

Core length/diameter ratio $= 1.2$.

$$\text{Estimated potential cube strength} = \frac{3.25}{\left(1.5+\frac{1}{1.2}\right)} \times 20.5 \quad \text{for a horizontal core.}$$

$$= 28.5\ \text{N/mm}^2$$

Reinforcement correction factor $= 1.09$.

Potential density of concrete $= 2350\ \text{kg/m}^3$ (mean value from cubes).

$$\text{Excess voidage} = \left(\frac{2350-2320}{2350-500}\right) \times 100\%$$

$$= 1.6\%.$$

Strength multiplying factor $= 1.14$ (Figure 5.4).

Corrected potential cube strength $= 28.5 \times 1.09 \times 1.14$

$$= 35.5\ \text{N/mm}^2.$$

(*Note:* an accuracy cannot be realistically quoted for a single result but the mean estimated potential cube strength from a group of at least four cores may be quoted to $\pm 15\%$ subject to a procedure described by Technical Report No. 11 (4) to eliminate abnormally low results.)

Procedure to eliminate abnormal results (*Ref. 4*)

This requires that for n cores (where n is at least 4) the lowest value is separated and the value t calculated from

$$t = \frac{\text{mean of remainder} - \text{lowest}}{\frac{\text{mean of remainder} \times 6}{100} \times \sqrt{1+\frac{1}{n-1}}}.$$

The value t is then compared with table B1, and if greater than the value in column A, corresponding to the total number of cores n, the lowest core result is discarded if there is any evidence of abnormality in relation to the others.

Table B1

No. of cores (n)	t *A*	t *B*
4	2.9	4.3
5	2.4	3.2
6	2.1	2.8
7	2.0	2.6
8	1.9	2.5

This may be in terms of location, reinforcement, compaction, cracks or drilling damage. If the value of t is greater than that in column B, the lowest result is discarded irrespective of other considerations. The mean value obtained from the remaining $(n-1)$ cores is then taken as the estimated potential strength.

If an abnormally high result is obtained, although this is less likely, the same procedure can be adopted but substituting the highest value for the lowest. Applying this procedure to the results used in section A1 (p. 188), i.e., 4 cores with potential strengths 27, 29, 32, 35 N/mm^2,

$$t = \frac{32-27}{\frac{32 \times 6}{100}\sqrt{1+\frac{1}{3}}}$$

$$= 2.26$$

This is less than the value 2.9 from column A, Table B1, and the quoted mean and $\pm 15\%$ accuracy may be considered valid.

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Paper 2

"Determining concrete strength by using small-diameter cores"

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Determining concrete strength by using small-diameter cores

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SYNOPSIS

Estimates of concrete strength are often made from compression tests on cores which have a diameter considerably less than the recommended 100 mm. The paper examines the results of a laboratory investigation in which 44 mm diameter cores were cut and tested. The influence of both specimen and aggregate size upon height/diameter ratio and orientation effects is examined, and observed relationships between core strengths and measured control cube strengths are compared with those normally used for larger cores. It is shown that core strength is affected by both specimen size and aggregate size, and it is proposed that conversion to corresponding cube strengths should take this into account. The variability of results is also assessed in relation to the above factors, and an estimate made of the accuracy of predicted actual cube strength that is likely to be achieved from testing cores of this size.

Introduction

The need for compressive testing of cores to yield an estimate of the strength of suspect concrete is well established, and recommendations for such tests are contained in BS 1881: Part 4:1970[1]. Also, the Concrete Society Technical Report No. 11 of 1976[2] provides considerably more detailed evidence and recommendations for both testing and interpretation of results. Both the above documents are based on cores of 150 or 100 diameter. However, it is frequently found to be totally impracticable to obtain cores of this diameter with the required minimum height/diameter of 1·0. This may be due either to limitations of member dimensions or to critical reinforcement locations, and is especially relevant to prestressed concrete construction. Consequently, cores of a considerably smaller diameter are often used, despite very limited evidence of their reliability. It was, therefore, considered worth while to undertake a testing programme with small-diameter cores to investigate the influence of a number of common variables, and also to examine the results in relation to recommended procedures for 'large' cores.

Aims and scope of investigation

The general problems of core testing are well known, and the factors which influence the relationship between the strength of a core and the corresponding Actual Cube Strength are described fully in Part 5 of the Concrete Society Report[2], which also summarizes past research on the topic, including that on 'small' cores.

The principal factors which may cause differences in behaviour between 'small' and 'large' cores are as follows.

(1) *Effects of size of specimen*

Research by Neville[3] and many other investigators suggests that measured concrete strength generally increases as the size of the test specimen decreases, and that results tend to be more variable with small specimens. The latter effect has been confirmed for core specimens by Henzel and Freitag[4].

(2) *Effects of cutting*

The ratio of cut surface area to volume increases as core diameter decreases, hence the potential influence of drilling damage upon measured strength will be greater with 'small' cores.

(3) *Relationship between size of aggregate and diameter of core*

This will be more critical with 'small' cores, which may commonly have aggregate-size/core-diameter ratios in excess of the suggested limit of 1/3, which is recommended by the American[5], German[6] and Australian[7] Standards. Where the aggregate particles are large in relation to the size of the specimen, the effects of any aggregate loosened by cutting will be increased. Furthermore, the homogeneity of the

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material in the test specimen is effectively reduced in comparison with a larger specimen, and this may influence the internal failure characteristics.

Since they are interrelated, differentiation between individual effects will be difficult. For example, increased strength attributed to small specimen size may be offset by a reduction due to greater cutting effects. Cutting damage may furthermore be influenced by aggregate characteristics, and each may separately influence the mode of failure and the variability of results.

Kesler[8] has shown that concrete strength is a further factor which may influence the behaviour of a core, and it is possible that this also may affect the relative behaviour of 'small' and 'large' cores. The complexity of these problems contributed to the decision to confine the investigation to a single 'small' core size, and 44 mm diameter was chosen as being typical of 'small' cores used in practice.

Advantage was taken of laboratory conditions to standardize, as far as possible, many of the variables which are known to affect the measured strength of a core (i.e. compaction, reinforcement, drilling technique, capping, moisture condition and testing technique). The two variables which will have a major effect upon the measured strength of a 'large' core are the height/diameter ratio (h/d) and the orientation of the direction of cutting and testing relative to that of casting. Attention was, therefore, concentrated upon examining the effects of concrete strength and aggregate size upon these relationships and the subsequent correlations between core strength and cube strength for the 'small' cores.

The differences between Potential Strength and Actual Strength of concrete are well documented[2], and have not been considered in this investigation, since the relationship is not directly influenced by the size of test specimen.

Details of test programme

A total of 23 mixes was used in the investigation, covering a range of measured cube strengths between 10 and 82 N/mm², and these are listed in Table 1. Portland cements were generally used, although three mixes were of high alumina cement concrete, and coarse aggregates were 10 mm or 20 mm maximum size irregular gravels. In all cases, 100 × 100 × 500 mm unreinforced prisms were cast and cured in the laboratory, together with at least four 100 mm control cubes. Nominal 44 mm diameter cores were cut from the prisms by using a diamond-tipped core-cutter, and were trimmed and capped with a thickness of up to 2 mm of high alumina cement mortar to give over-all height/diameter ratios between 1·0 and 2·0.

TABLE 1: **Summary of test specimens.**

Mix No.	Measured cube strength (N/mm²)	Maximum aggregate size (mm)	Type of cement	h/d of cores	Total number of cores, Orientation* H	Total number of cores, Orientation* V
1	82	10	HA	1·2, 2·0	6	6
2	82	20	HA	1·2, 1·7, 2·0	6	6
3	75	20	HA	1·2, 1·7, 2·0	—	24
4	23	20	OP	1·2, 1·6, 2·0	—	24
5	41	20	OP	1·2, 1·6, 2·0	—	24
6	75	20	OP	1·2, 1·6, 2·0	—	24
7	34	20	RHP	1·4, 1·8, 2·0	—	24
8	34	20	RHP	1·2, 2·0	8	8
9	21	20	RHP	1·2, 2·0	—	16
10	68	20	RHP	1·2, 2·0	—	16
11	33	10	RHP	1·5, 1·8, 2·0	—	20
12	33	10	RHP	1·2, 2·0	8	8
13	23	10	RHP	1·5, 1·8, 2·0	—	24
14	23	10	RHP	1·2, 2·0	8	8
15	10	10	RHP	1·2, 1·8, 2·0	—	16
16	45	10	RHP	2·0	4	—
17	36	10	RHP	2·0	4	—
18	31	10	RHP	2·0	4	—
19	18	10	RHP	2·0	4	—
20	43	20	RHP	2·0	4	—
21	33	20	RHP	2·0	4	—
22	36	20	RHP	2·0	4	—
23	24	20	RHP	2·0	4	—
				Totals	68	248

*See Figure 1.

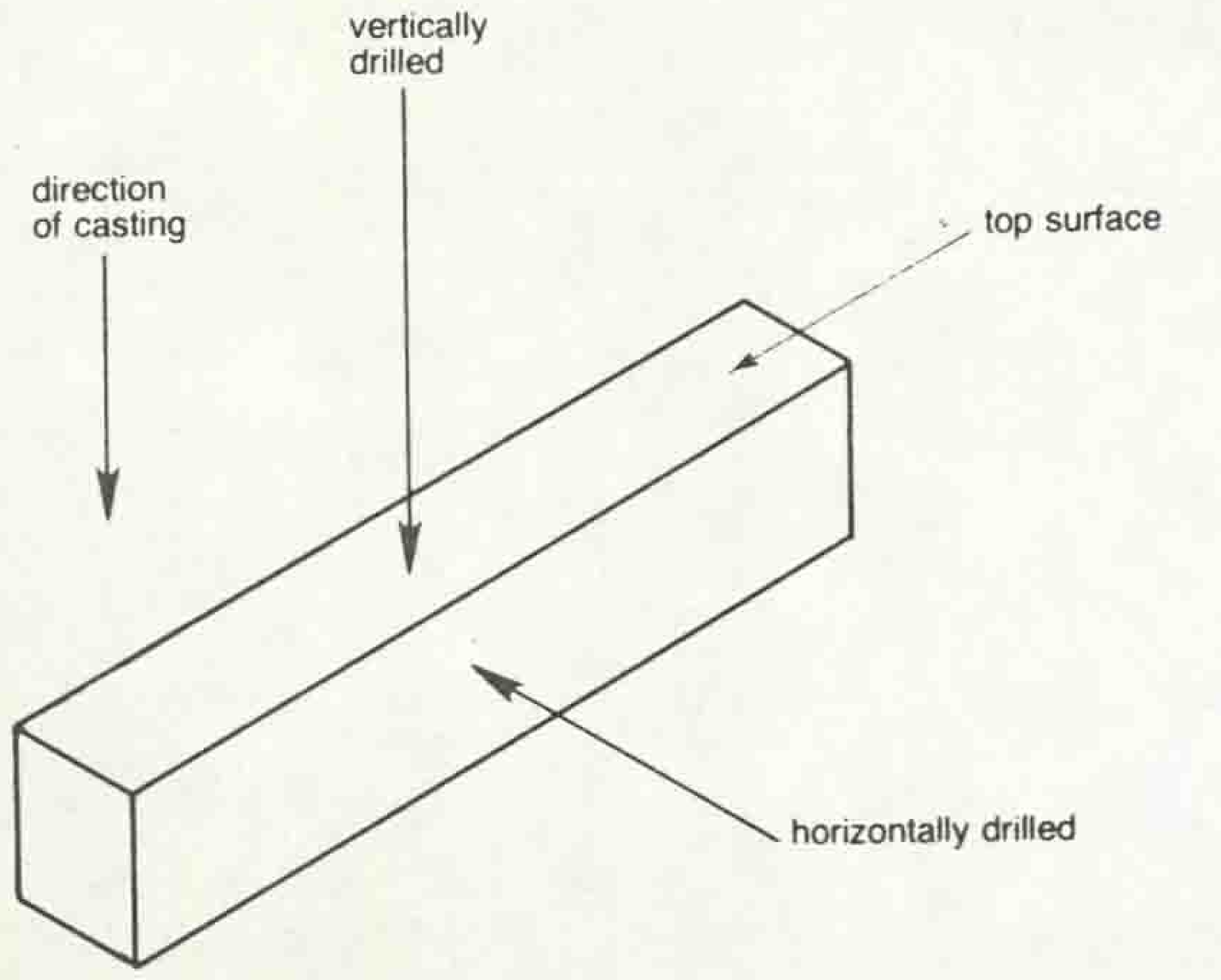

Figure 1: Drilling orientation in relation to prism specimen.

Although the majority of cores were cut in a 'vertical' direction as defined in Figure 1, a number were cut 'horizontally' relative to the direction of casting.

Cores were not cut until long enough after casting to allow effective stabilization of concrete strength, and thus minimize variations due to curing differences between cores, after cutting, and cubes. Minimum ages of 14 days for rapid-hardening and 28 days for ordinary Portland cements were considered adequate for this purpose.

A specially designed frame was used to hold the prisms during cutting, to limit relative movement between the specimen and rig, and cores were stored under water for at least 48 h prior to compressive testing, to standardize moisture contents. Since the cubes were in every case cured with the prisms and cores, the cube strengths obtained do not represent standard 28 day strengths but are actual cube strengths which relate as closely as possible to the strength of the concrete in the cores at the time of testing.

The cores were tested in compression immediately after removal from water, with loading applied at a rate of 15 N/mm² min, as recommended by BS 1881, Part 4[1].

Those with an anticipated measured strength of 40 N/mm² or less were tested in a 6·6 tonne manually operated Denison testing machine, whilst a 100 tonne capacity Avery machine was used for stronger specimens. The failure of each core was examined visually to ensure that there was no significant cracking or damage to the caps and that the failure mode was symmetrical. Cases where diagonal shear failure occurred were accepted for long cores, as recommended by the Concrete Society Report[2], but were rejected if h/d was less than 1·4.

Analysis and discussion of experimental results

INFLUENCE OF HEIGHT/DIAMETER RATIO (h/d)

For each mix, the average values of measured core strengths, with a specific orientation, were compared for each different value of h/d and expressed in terms of a core with $h/d = 2{\cdot}0$. The averages were all based on the results of at least four individual similar cores, although a number of prism specimens were often involved and thus the variability of results will be partly due to variations of concrete between specimens. The results of this analysis are illustrated in Figure 2, and show a very large scatter. Nevertheless an over-all least-squares regression line $K = 0{\cdot}54 + 0{\cdot}23\ (h/d)$ is shown. Although a distinction has been made between the two basic types of cement in this Figure, separate analysis does not yield any significant influence of this variable. Similarly, although not illustrated, the effect of drilling orientation was also found not to be significant in this respect.

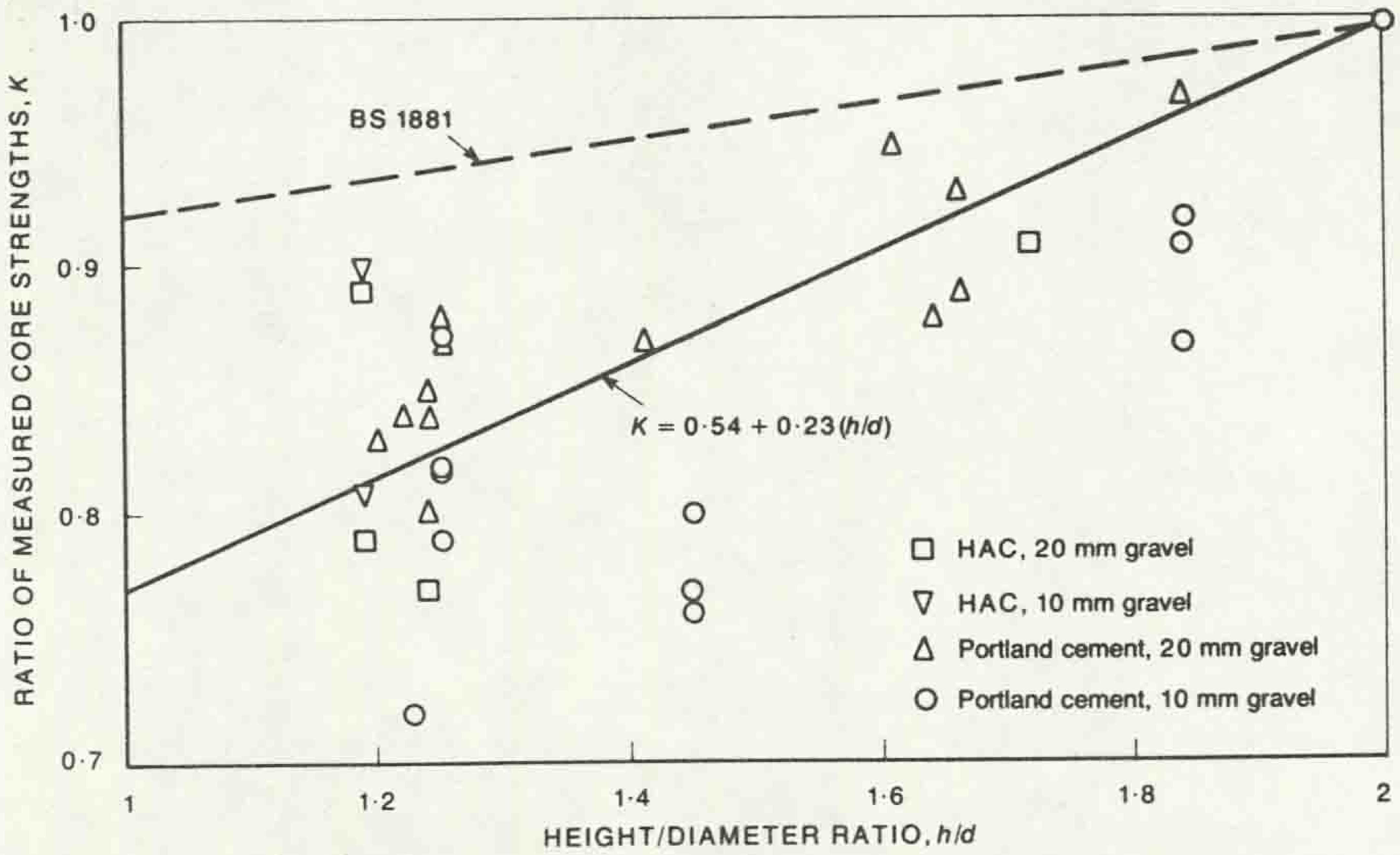

Figure 2: Influence of height/diameter ratio.

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Maximum aggregate size is also indicated in Figure 2, and separate analyses give $K = 0{\cdot}57 + 0{\cdot}215\ (h/d)$ for 20 mm, and $K = 0{\cdot}50 + 0{\cdot}25\ (h/d)$ for 10 mm results. The scatter must limit the reliability of these expressions; however, the corresponding correction factors to a standard $h/d = 2{\cdot}0$ are summarized in Table 2 and compared with the recommendations of references 1 and 2. Although the strength of 10 mm aggregate cores is apparently more influenced by variations of h/d than is that of the 20 mm aggregate specimens, it will be seen that the scatter on observed values is so great that discrepancies in corrected core strength due to adopting an average correction factor which is independent of aggregate size will be relatively unimportant.

VARIABILITY OF TEST RESULTS

Coefficients of variation for each set of 'identical' vertically drilled cores of Portland cement concrete are summarized in Table 3. It will be observed that there is no significant change in variability of results between the two extreme values of h/d, irrespective of aggregate size. Results for 10 mm aggregate cores show a slightly greater scatter than 20 mm results. However, this may be misleading in view of the differences obtained for coefficient of variation of measured cube strength between mixes (Table 4). Whilst this can be due to a combination of factors[9], since similar casting and testing procedures were used it is likely to reflect differing variability of the concrete within individual batches. Thus the ratio (coefficient of variation of cores)/(coefficient of variation of cubes) is more likely to represent differences in behaviour between the various types of concrete due to core testing alone.

In Table 4, average coefficients of variation are shown according to cement type, aggregate size and core orientation. Examination of the core/cube ratio suggests that 20 mm aggregate cores exhibit a greater variability due to cutting and testing than 10 mm aggregate cores. The scatter of results is such that this is not proved conclusively: nevertheless, further weight is given to this view by the appreciable differences in standard deviations of this ratio between 10 mm and 20 mm aggregates. Orientation seems to have little influence upon the variability of core results. However, it can be seen from Table 4 that cores from the high alumina cement mixes appear to be more variable than those from the Portland cement mixes.

TABLE 2: **(h/d) correction factors.**

		h/d 1·0	h/d 2·0
'Small' cores	10 mm gravel	0·75	1·0
	20 mm gravel	0·785	1·0
	Experimental average	0·77	1·0
'Large' cores	Concrete Society[2]	0·80	1·0
	BS 1881, Part 4[1]	0·92	1·0

TABLE 3: **Effect of (h/d) upon coefficient of variation of measured core strength (Portland cement concrete).**

Aggregate	Average coefficient of variation ('vertical' cores) (%): h/d = 1·2	h/d = 1·4–1·8	h/d = 2·0	Over-all mean	95% confidence limits
20 mm gravel (7 mixes)	7·8	7·3	8·1	7·5	± 4·7
10 mm gravel (5 mixes)	7·3	7·1	9·8	8·8	± 6·0

TABLE 4: **Summary of coefficients of variation of test results.**

Type of cement and size of aggregate	Orientation of core*	Number of mixes	Average coefficient of variation (%): Cores	Cubes	Core/cube ratio	Standard deviation of core/cube ratio
PORTLAND CEMENT						
20 mm gravel	V	7	7·5	5·5	1·8	1·3
	H	5	5·8	3·4	1·7	1·2
10 mm gravel	V	5	8·8	7·0	1·3	0·6
	H	6	9·5	6·4	1·3	0·5
HIGH ALUMINA CEMENT						
20 and 10 mm gravel	V and H	3	10·8	4·1	2·7	1·2

*See Figure 1.

MEASURED STRENGTHS

In practice, strength assessments are likely to be based on at least four similar cores, and the results of this investigation have generally been assessed on this basis. Nevertheless, to illustrate the variability of individual results, Figure 3 shows values obtained for single 'horizontal' Portland cement cores with 10 mm aggregate.

Further comparisons of measured core and cube strengths for Portland cement concrete are shown in Figures 4 and 5, for 'horizontally' drilled and 'vertically' drilled cores respectively. In each case, core strengths have been corrected to a standard $h/d = 2{\cdot}0$ by using the relationship $K = 0{\cdot}54 + 0{\cdot}23\ (h/d)$ established above, and each point represents the mean of four individual similar cores. A distinction has been made between the two maximum aggregate sizes, and least-squares straight-line relationships are

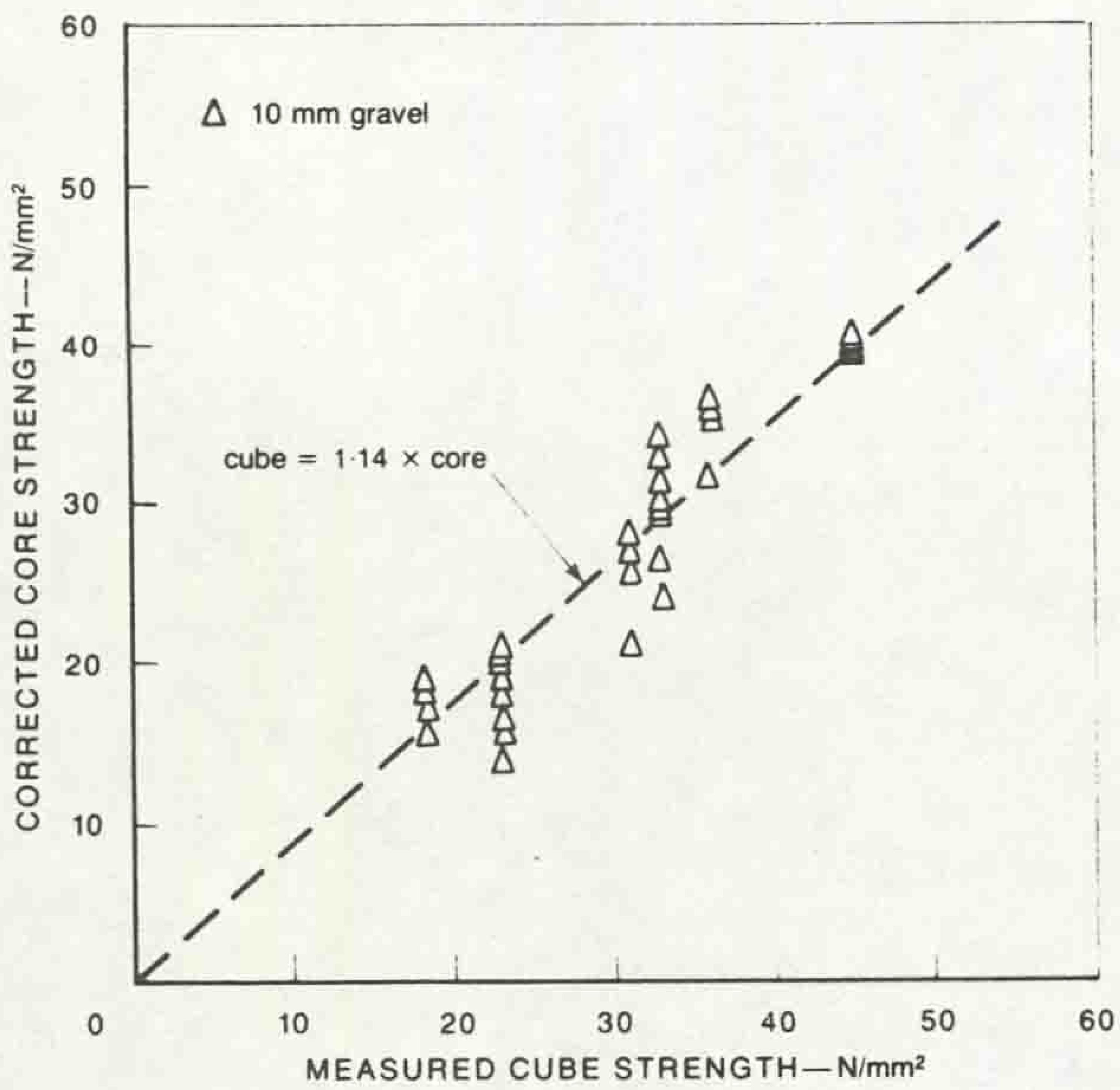

Figure 3: Relationship between corrected core strength (single cores, h/d = 20) and measured cube strength (Portland cement concrete) for 'horizontal' cores.

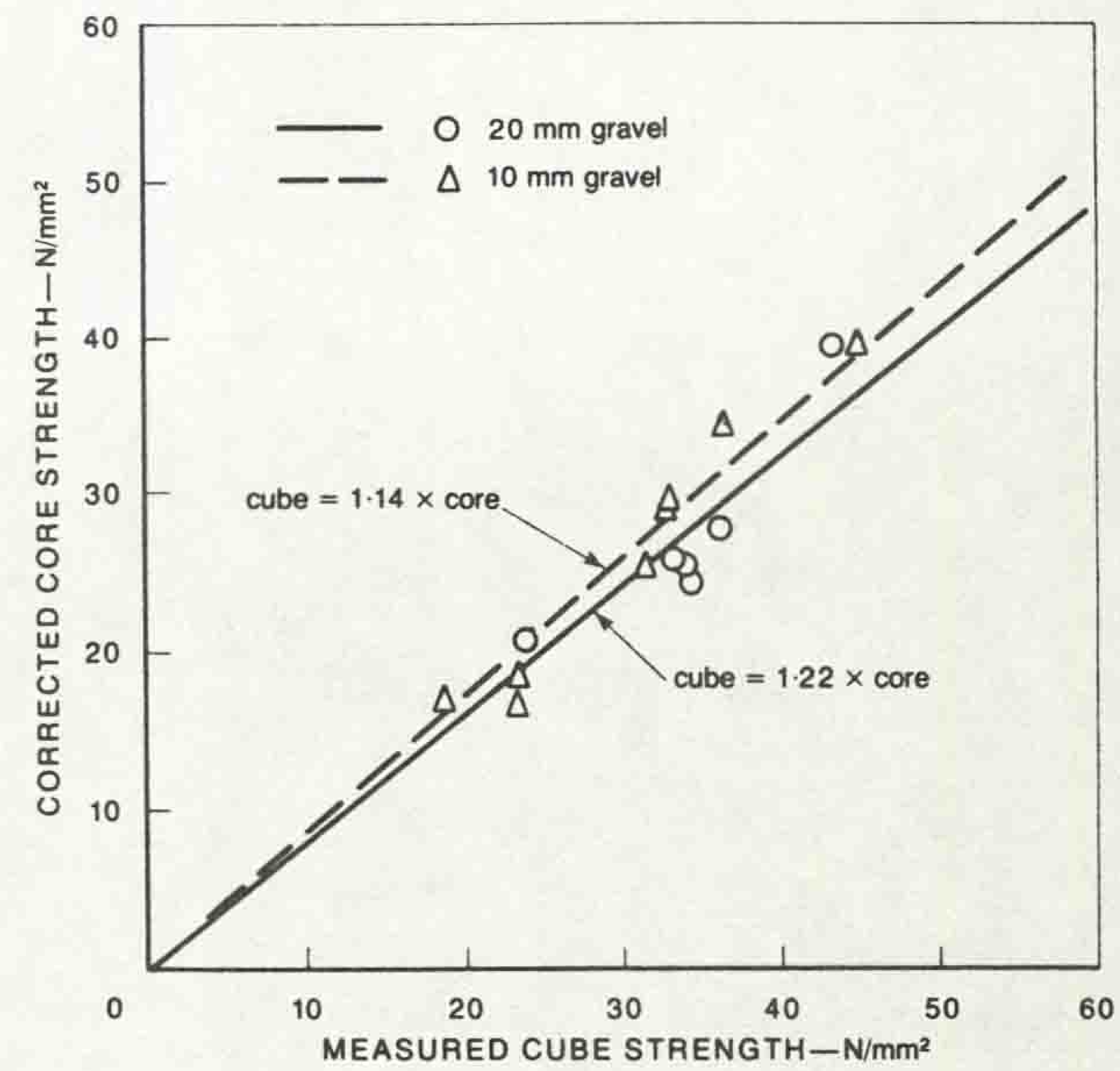

Figure 4: Relationship between corrected core strength (mean of 4, h/d = 2·0) and measured cube strength (Portland cement concrete) for 'horizontal' cores.

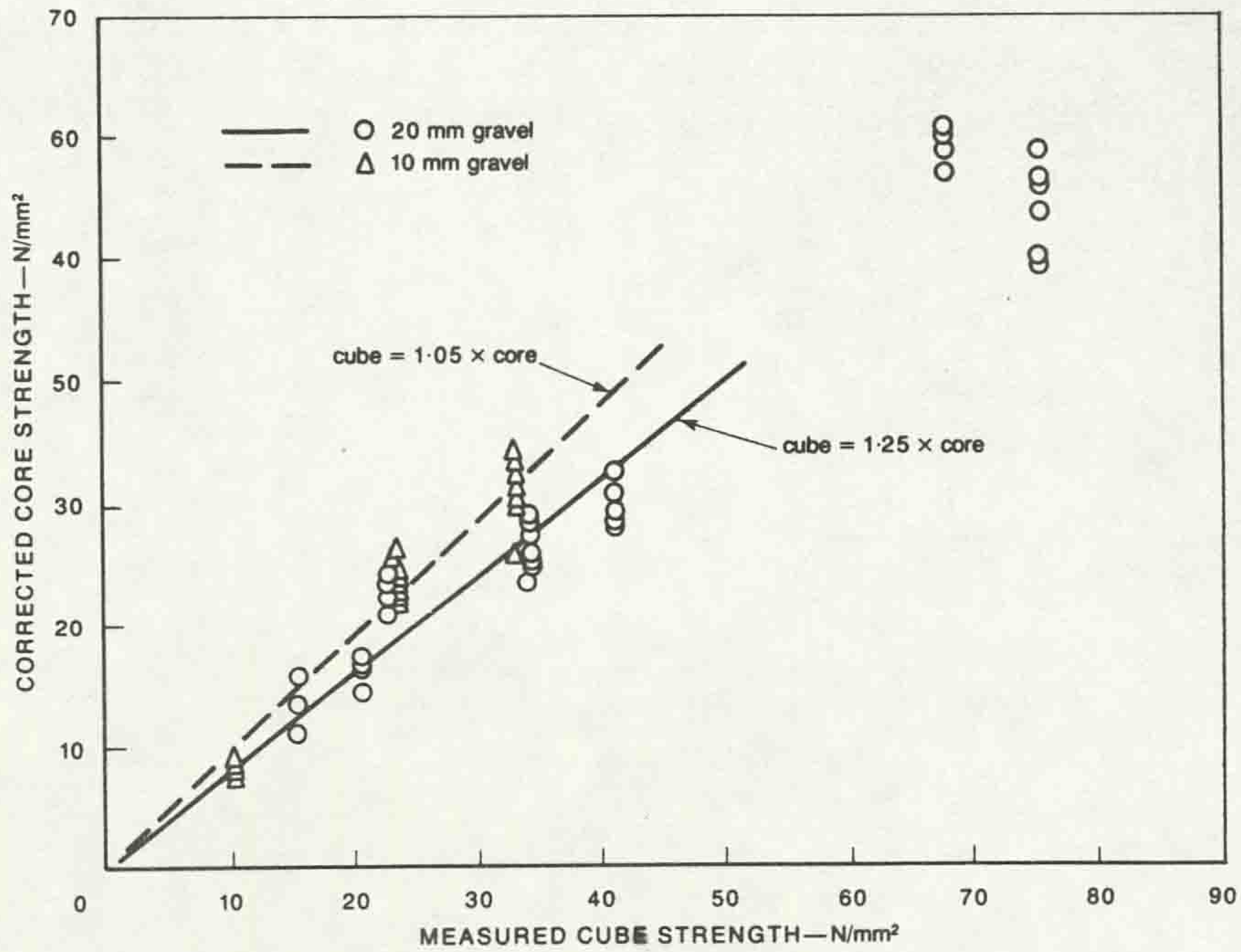

Figure 5: Relationship between corrected core strength (mean of 4, h/d = 2·0) and measured cube strength (Portland cement concrete) for 'vertical' cores.

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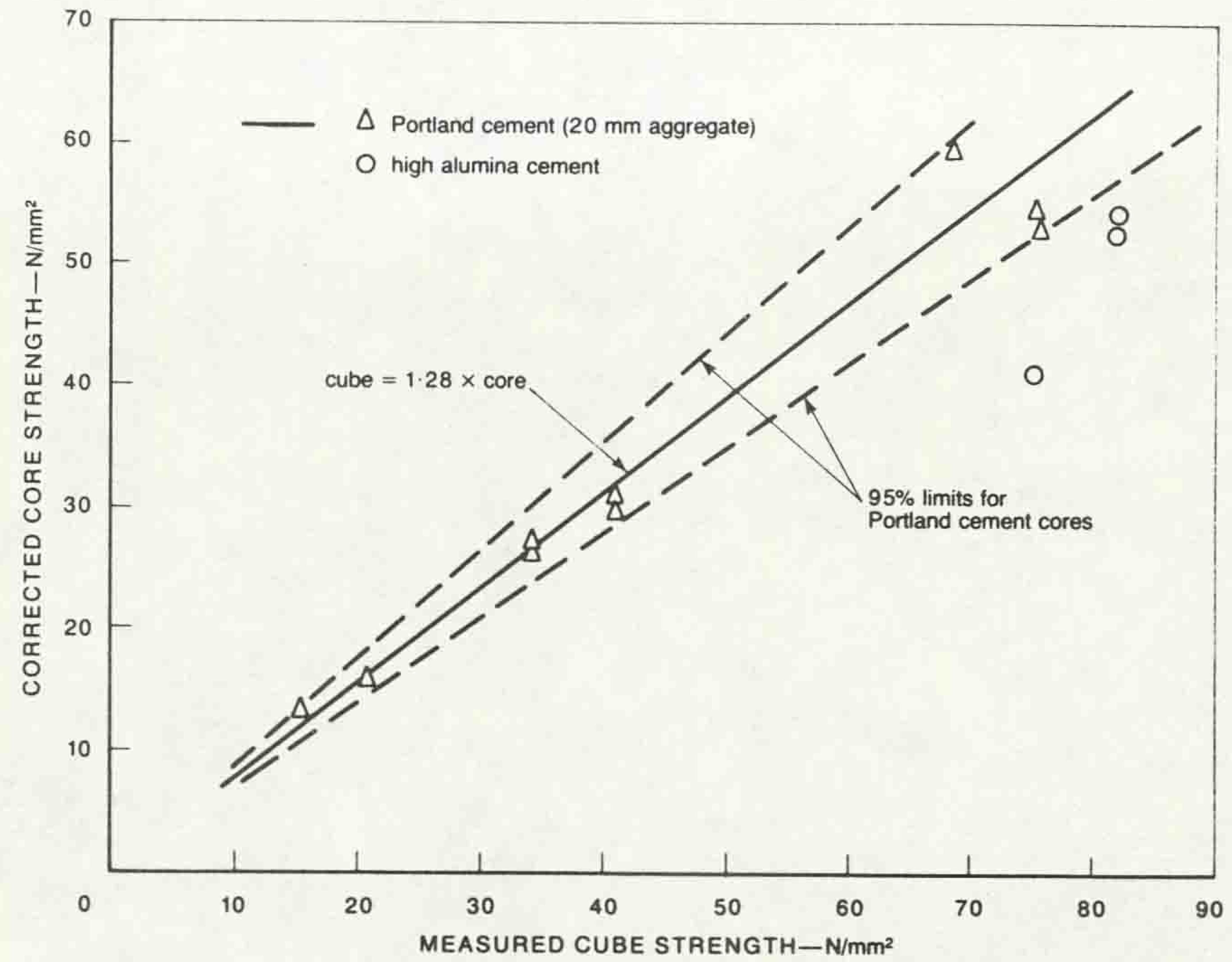

Figure 6: Relationship between corrected core strength (mean of 12, h/d = 2·0) and measured cube strength for 'vertical' cores.

shown for each size. These cube/core strength correlations are summarized in Table 5, together with the corresponding 95% confidence limits on calculated cube strengths. To permit realistic comparison of results, mixes with a very high cube strength (<50 N/mm²) have not been included in calculating these relationships. It was found, nevertheless, that for the vertically drilled 20 mm aggregate cores alone, the conversion factor is only marginally changed from 1·25 to 1·28 with 95% limits of ±22% when the higher strengths are included. Figure 6 is based on means of 12 'vertical' 20 mm aggregate cores, and shows the corresponding 95% confidence limits which are reduced to ±12% in that case. The results for HAC mixes are also plotted in Figure 6 and show that the core strengths are significantly lower than would be expected for similar Portland cement cores.

General observations, and comparison of results with existing recommendations

The results clearly demonstrate that for 'small' cores (*h*/*d*) effects are considerably greater than those suggested by BS 1881[1]. Comparison of the experimental results with the more recent Concrete Society recommendations (Table 2) suggests that 'small' cores may be marginally more sensitive to *h*/*d* variations than large cores, but there is no evidence that the over-all variability of core results is affected by the *h*/*d* ratio used.

The average coefficient of variation for core results

TABLE 5: **Cube/corrected-core conversion factors (cube strength < 50 N/mm²).**

Orientation	Maximum aggregate size	Experimental results 10 mm	Experimental results 20 mm	Experimental results Combined	Concrete Society recommendations[2] (for large cores)
'Vertical'	Conversion factor	1·05	1·25	1·15	1·15
	95% limits on predicted cube strength (mean of 4 cores)	±17%	±23%	±23%	±6%
'Horizontal'	Conversion factor	1·14	1·22	1·17	1·25
	95% limits on predicted cube strength (mean of 4 cores)	±15%	±17%	±17%	±6%

of approximately 8% compares well with that expected[2], although the range is large and no significant difference can be detected between 'horizontal' and 'vertical' cores. The indication that 20 mm aggregate cores may be more variable than those with 10 mm aggregate is likely to reflect weakening of the cut surface. Although damage may not be visible, any pieces of aggregate which are cut, and thus only partially embedded in the matrix, may influence the failure of the core and it is to be expected that this will become more significant as the ratio of aggregate size to core diameter increases. HAC cores show a particularly high variability, which may be due to deterioration of the core matrix as a result of conversion[10] being accelerated by drilling and subsequent curing of the cores. This view is supported by the lower than expected core strengths obtained (Figure 6). Measurements of percentage conversion were not made, but it is expected that this would have been fairly low in the cube and prism specimens, and the concrete may thus have been vulnerable to the heat generated in the surface zone by the drilling. These results, although few in number, suggest caution where HAC is involved, and it would seem that this problem should be examined more fully.

The strength of cores with 10 mm aggregate follows the anticipated pattern, 'horizontal' cores being approximately 10% weaker relative to cubes than 'vertical' cores. However, the corrected core strengths are 10% stronger than would be anticipated from the Concrete Society recommendations for 'large' cores (Table 5). This discrepancy will be further increased by a small amount due to the differences between *h/d* correction factors (Table 2), but would seem to be primarily associated with the reduction in specimen size.

The 20 mm aggregate cores are considerably weaker relative to cubes than 10 mm aggregate cores, which tends to confirm the anticipated influence of the (aggregate size)/(core diameter) ratio discussed above. This is further supported by the greater variability of results, and is reflected in the wider band of 95% confidence limits on predicted cube strength. The anticipated orientation effects are not apparent for 20 mm aggregate concretes, but the strengths obtained from vertically drilled cores are lower and show a greater scatter than would be expected. This may possibly be attributed to greater variations of the larger aggregate concrete near 'top' surfaces, even in laboratory specimens, and serves to emphasize the need in practice to avoid taking cores from concrete from the top of pours. In the light of these results it is suggested that, for 'small' cores of this diameter, 10 mm and 20 mm maximum aggregates should be treated separately when attempting to convert core results to cube strengths. If the experimental correlations for 'vertical' or 'horizontal' cores are used as appropriate, it is unlikely that 95% confidence limits on actual cube strength will be better than those indicated in Table 5. These are considerably worse than the values suggested for 'large' cores, being at least three times the Concrete Society[2] value of $\pm(12/\sqrt{n})\%$, where n is the number of cores averaged. Direct application of the Concrete Society recommendations[2] to this set of results for 'small' cores would have predicted actual cube strengths which were up to 30% different from the measured value. A similar analysis based on BS 1881, Part 4, yielded estimated cube strengths which are on average 12% too high, and range between 50% above and 20% below the measured value. Although only one typical 'small' core size has been used here, the results demonstrate clearly that existing methods for obtaining cube strengths from 'large' diameter cores must be treated with great caution when smaller diameters are used.

From information given by Warren[9] it can be shown that, on the basis of the cube results, the concrete used in this investigation is 'good' or 'average' in terms of variability. Reduction of quality to 'mediocre', as may occur on site, could possibly widen the range of 95% confidence limits on actual cube strengths predicted from means of four site cores by an estimated ±5%. This assumes no difference in cutting procedures between site and laboratory, which may be optimistic. In interpreting site results, other factors such as voids and reinforcement must also be considered, and these may further influence strength predictions. Caution must thus be exercised when attempting to assess the accuracy of actual concrete cube strength predictions from site-cut cores, and the range of confidence limits with cores of this size is unlikely to be smaller than suggested by this laboratory investigation, even under good conditions.

The sensitivity of 'small' cores to the many factors outlined above must mean that the variability of results is likely to be so large that the use of such cores to assess the strength of in situ concrete is of doubtful value in many practical situations.

Conclusions

For 'small' cores of 44 mm diameter, the following conclusions may be drawn.

(1) The effect of height/diameter ratio is considerably greater than that indicated by BS 1881, Part 4[1], and is closer to, but still marginally greater than that proposed by the Concrete Society Report[2] relating to 'large' cores.

(2) The average coefficient of variation associated with a set of Portland cement concrete cores drilled and tested under laboratory conditions is of the order of 8%, and is not significantly influenced by height/diameter ratio.

(3) Where 10 mm maximum aggregates are used, the relative orientations of casting and testing have an

effect upon measured strength which is similar to that anticipated for larger cores.

(4) The measured strength of cores with 10 mm maximum aggregate size is approximately 10% greater relative to the actual cube strength than would be expected for cores of 100 or 150 mm diameter. This was not found to apply with 20 mm aggregates for which 'vertically' drilled cores were approximately 10% weaker than anticipated.

(5) The strengths of cores from 'uncoverted' HAC concrete may be lower in relation to cubes, and more variable, than would be expected for similar Portland cement cores.

(6) The sensitivity of 'small' cores to many factors of mix properties and testing procedure leads to a high variability of strength predictions, and methods used for estimating cube strengths from 'large' cores cannot be relied upon under these circumstances.

(7) Prediction of actual cube strength should account for both orientation of cores and aggregate size, in accordance with Table 5. In this event, for a set of n results the 95% confidence limits are unlikely to be better than $\pm (36/\sqrt{n})\%$ under laboratory conditions. The number of additional influences associated with site conditions suggests that the use of 'small' cores to assess the equivalent cube strength of in situ concrete may be of doubtful value.

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Contributions discussing the above paper should be in the hands of the Editor not later than 31 December 1979.

Discussion on papers published in the

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Volume 31, Number 107 : June 1979

Determining concrete strength by using small-diameter cores*

J. H. Bungey

Contribution by R. K. Lewis

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The Australian Standard for the securing and testing of cores from hardened concrete[1] requires that the diameter of the core shall be not less than 75 mm. I believe that this limitation is justified and agree with Mr Bungey when he says, "... the variability of results is likely to be so large that the use of such cores [44 mm diameter] to assess the strength of in situ concrete is of doubtful value in many practical situations". Nevertheless, there are times when cores having a diameter less than 75 mm must be taken and the work by Bungey will be of value in interpreting the results.

The Australian Standard also places a limitation on the ratio of the core diameter to the nominal maximum size of the aggregate at a value of 3. This follows the work of Rechardt[2], who plotted the increase in standard deviation of cores over the dispersion in strength of separately cast cubes, against the ratio of the core diameter to the maximum particle size. Little difference in the dispersion of the standard deviation occurred, provided the ratio was greater than 3. Bungey states that the 20 mm aggregate cores ($d/A_{max} = 2{\cdot}2$) exhibit a greater variability than the 10 mm aggregate ($d/A_{max} = 4{\cdot}4$), although this is not proved conclusively. It would appear therefore that the limitation of the ratio to be not less than 3 should still apply.

In our study at the Division of Building Research on the compressive strength of cores[3], we noted that the core strengths were always less than the strength obtained from standard cylinders, even when good curing was provided. Also that the smaller the core diameter, the lower the compressive strength. To estimate the strength of 150 mm diameter cores from smaller cores, it was necessary to multiply the 100 mm diameter core by 1·07 and the 75 mm diameter core by 1·11.

In subsequent work[4] it was shown that these strength differences from cores of various sizes were not due to the effect of the diameter, but rather they were related to a strength gradient through the concrete test slab. Where a strength gradient between top and bottom portions of the slab existed, small-diameter core specimens taken from the top surface reflected the strength at the top, whilst larger-diameter cores reflected the concrete strength at a lower level – all cores being secured from the top surface. Where no gradient existed, the observed strengths of the cores of different diameters were not significantly different, which supports the findings by Petersons[5].

The above is an important factor when small-diameter cores are taken in practice. Under these circumstances, it is necessary to give due consider-

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*Pages 91 to 98 of *MCR* 107.

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ation to the location of the core. If we are interested in the surface strength of a slab, smaller-diameter cores will give a better estimate than larger cores. If we persist with small-diameter cores for an estimate of the strength of the concrete mass, they should be taken near the centre of that mass.

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Contribution by S. A. W. Bowman

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The specification for a foundation contract in Tsuen Wan, one of the Hong Kong new towns, included a requirement for small-diameter cores to be taken from large cast-in-place piles after casting. The cores were to be checked visually for evidence of flaws, and crushed to confirm the strengths achieved by concrete test cubes. Objections were raised to the use of small-diameter cores for strength testing, on the grounds that they would not give a reliable indication of strength, and that to penalize the contractor on such unreliable evidence would not be proper.

At the same time, a proposal to widen a flyover in Kowloon led to cores of 150 mm diameter being taken to assess the actual strength of the concrete in the flyover. In order to compare the variability of results from large- and small-diameter cores taken from an actual structure, 50 mm diameter cores were taken as well, from positions as near as possible to the 150 mm cores.

Twenty-five 50 mm cores and twenty-four 150 mm cores were taken. Results from the 50 mm cores gave a mean strength of 44·7 N/mm² and a coefficient of variation of 28·9%. The mean strength of the 150 mm cores was 41·4 N/mm², and the coefficient of variation was 19·5%. The greater variability of results from small-diameter cores was thus confirmed.

In spite of this, it was concluded that 50 mm cores, being relatively cheap and quick to obtain, should continue to be used, not only for visual checks, but also for strength tests on the concrete in cast-in-place piles. To allow for the variability of such tests, it was laid down that, if the measured compressive strength of any section of a 50 mm core fell below 87% of the characteristic strength of the concrete, the contractor should be given the option either of carrying out remedial works specified by the engineer, or of taking a confirmatory 150 mm core.

Reply by the author

I thank Mr Lewis and Mr Bowman for their interesting contributions.

Mr Lewis confirms the need for small cores in some circumstances and emphasizes the importance of the core-diameter/aggregate-size ratio. He also provides a number of useful references relating to this and other aspects of core performance. I would agree with him that ideally a limiting core-diameter/aggregate-size ratio of 3 should be maintained in order to contain the variability of results, but unfortunately this may not always be possible in practice.

Mr Lewis also highlights the particularly critical problem of strength variation throughout members, which affects not only cores but other methods of non-destructive strength evaluation, such as internal fracture and Windsor probe testing. I have recently observed a situation similar to that which he described when examining laboratory cast beams in which strength variations within the member dominated other variables.

An interesting example of a practical situation in which small core strength testing was found to be worth while is given by Mr Bowman. Although he does not indicate the size of aggregate, his figures for strength and coefficients of variation follow the pattern suggested by my work, and the difference in variability of results between site and laboratory work is well illustrated.

Both contributions confirm the importance of very careful planning and interpretation of any site core tests, and that this consideration is increased when the diameter is small.

Paper 3

"Surface Hardness Methods"

The Testing of Concrete in Structures

Chapter 2 Surrey University Press

1982

2 Surface hardness methods

One of many factors connected with the strength of concrete is its hardness. Efforts to measure the surface hardness of a mass of concrete were first recorded in the 1930's; tests were based on impacting the concrete surface by a specified mass activated by a standard amount of energy. Early methods involved measurement of the size of indentation caused by a steel ball either fixed to a pendulum or spring hammer, or fired from a standardized testing pistol (17). Later, however, the height of rebound of the mass from the surface was measured. Whilst it is difficult to justify a theoretical relationship between the measured values from any of these methods and the strength of a concrete, their value lies in the ability to establish empirical relationships between test results and strength. Unfortunately these are subject to many specific restrictions including concrete and member details, as well as equipment reliability and operator technique.

Indentation testing has received attention in Germany and the USSR as well as the United Kingdom, and although included in British Standard BS 4408 pt. 4 (18) has never become very popular. The rebound principle on the other hand is more widely accepted: the most popular equipment, the Schmidt Rebound Hammer, has been in use worldwide for many years.

2.1 Rebound test equipment and operation

A Swiss engineer, Ernst Schmidt (19), first developed a practicable rebound test hammer in the late 1940's and modern versions are based on this. Figure 2.1 shows the basic features of a typical type N hammer, which weighs less than 2 kg.

The spring-controlled hammer mass slides on a plunger within a tubular housing. The plunger retracts against a spring when pressed against the concrete surface and this spring is automatically released when fully tensioned, causing the hammer mass to impact against the concrete through the plunger. When the spring-controlled mass rebounds, it takes with it a

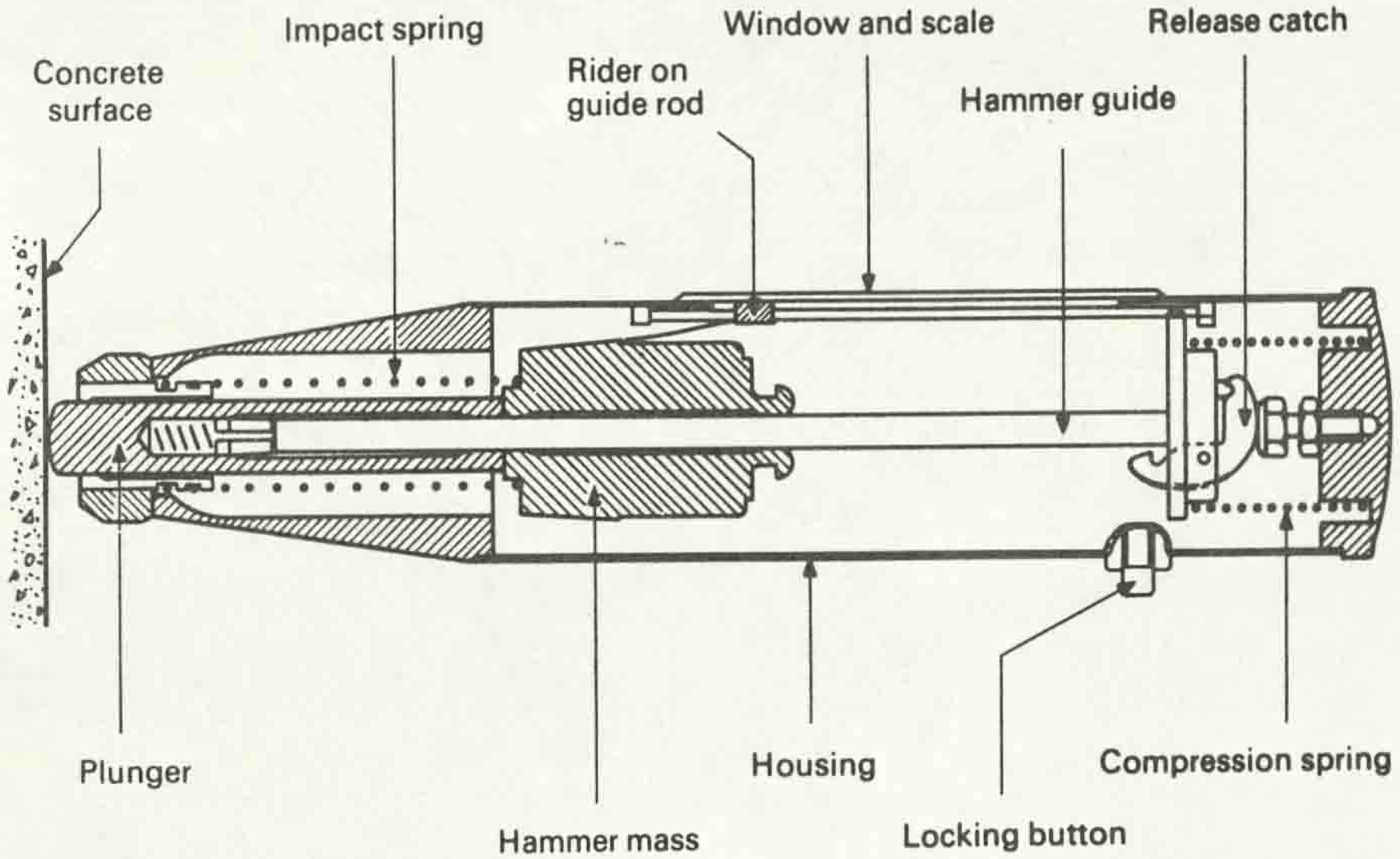

Figure 2.1 Typical rebound hammer.

rider which slides along a scale and is visible through a small window in the side of the casing. The rider can be held in position on the scale by depressing the locking button. The equipment is very simple to use (Figure 2.2), and may be operated either horizontally or vertically. The plunger is pressed strongly and steadily against the concrete at right angles to its surface, until the spring-loaded mass is triggered from its locked position. After the impact, the scale index is read whilst the hammer is still in the test position. Alternatively, the locking button may be pressed to enable the reading to be retained, or results can be recorded automatically by pen recorder. The scale reading is known as the rebound number, and is an arbitrary measure since it depends on the energy stored in the given spring and on the mass used. This version of the equipment is most commonly used, and is most suitable for concretes in the 20–60 N/mm^2 strength range. Other versions are available, and for lower strength concretes it is recommended that a pendulum type rebound hammer is used.

2.2 Procedure

The reading is very sensitive to local variations of the concrete, especially aggregate particles near to the surface. It is therefore necessary to take several readings at each test location, and to find their average. **BS 4408 pt. 4 (18)** recommends between 9 and 25 readings taken over an area not exceeding

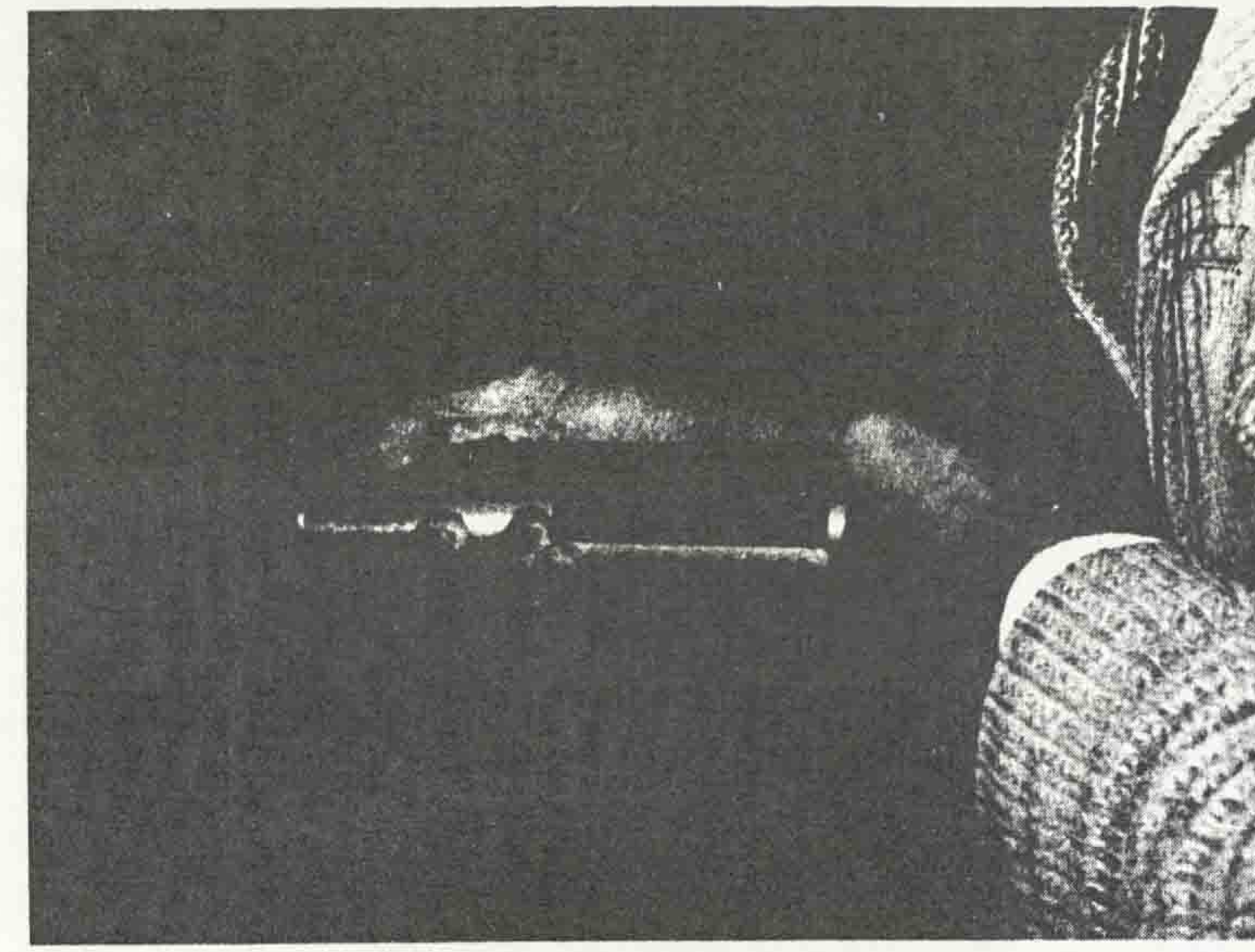

Figure 2.2 Schmidt Hammer in use.

300 mm square, with the impact points not less than 20 mm from each other or from an edge. The use of a grid to locate these points reduces operator bias. The surface must be smooth, clean and dry, and should preferably be formed, but if trowelled surfaces are unavoidable they should be rubbed smooth with the carborundum stone usually provided with the equipment. Loose material can be ground off, but areas which are rough from poor compaction, grout loss, spalling or tooling must be avoided since the results will be unreliable.

2.3 Theory, calibration and interpretation

The test is based on the principle that the rebound of an elastic mass depends on the hardness of the surface upon which it impinges, and in this case will provide information about a surface layer of the concrete defined as no more than 30 mm deep. The results give a measure of the relative hardness of this zone, and this cannot be directly related to any other property of the concrete. Energy is lost on impact due to localized crushing of the concrete and internal friction within the body of the concrete, and it is the latter, which is a function of the elastic properties of the concrete constituents, that makes theoretical evaluation of test results virtually impossible. Many factors influence results but must all be considered if rebound number is to be empirically related to strength.

2.3.1 *Factors influencing test results*

Results are significantly influenced by all the following

1. Mix characteristics
   (a) Cement type
   (b) Cement content
   (c) Coarse aggregate type.

2. Member characteristics
   (a) Mass
   (b) Compaction
   (c) Surface type
   (d) Age, rate of hardening and curing type
   (e) Surface carbonation
   (f) Moisture condition
   (g) Stress state and temperature.

Since each of these factors may affect the readings obtained, any attempts to compare or estimate concrete strength will be valid only if they are all standardized for the concrete under test and for the calibration specimens. These influences have different magnitudes, which are examined in detail below.

2.3.1.1 *Mix characteristics*

(a) *Cement type.* Variations in fineness of Portland cement are unlikely to be significant—their influence on strength correlation is less than 10%. Supersulphated cement, however, can be expected to yield strengths 50% lower than suggested by a Portland cement calibration, whilst high alumina cement concrete may be up to 100% stronger.

(b) *Cement content.* Changes in cement content do not result in corresponding changes in surface hardness. The combined influence of strength, workability and aggregate/cement proportions leads to a reduction of hardness relative to strength as the cement content increases (20). The error in estimated strength, however, is unlikely to exceed 10% from this cause for most mixes.

(c) *Coarse aggregate.* The influence of aggregate type and proportions can be considerable, since strength is governed by both paste and aggregate characteristics. The rebound number will be influenced more by the hardened paste. For example, crushed limestone may yield a rebound number 7 points lower than for a gravel concrete of similar strength (21). This would typically be equivalent to a strength difference of 6–7 N/mm^2. A particular aggregate type may also yield different rebound number/strength correlations

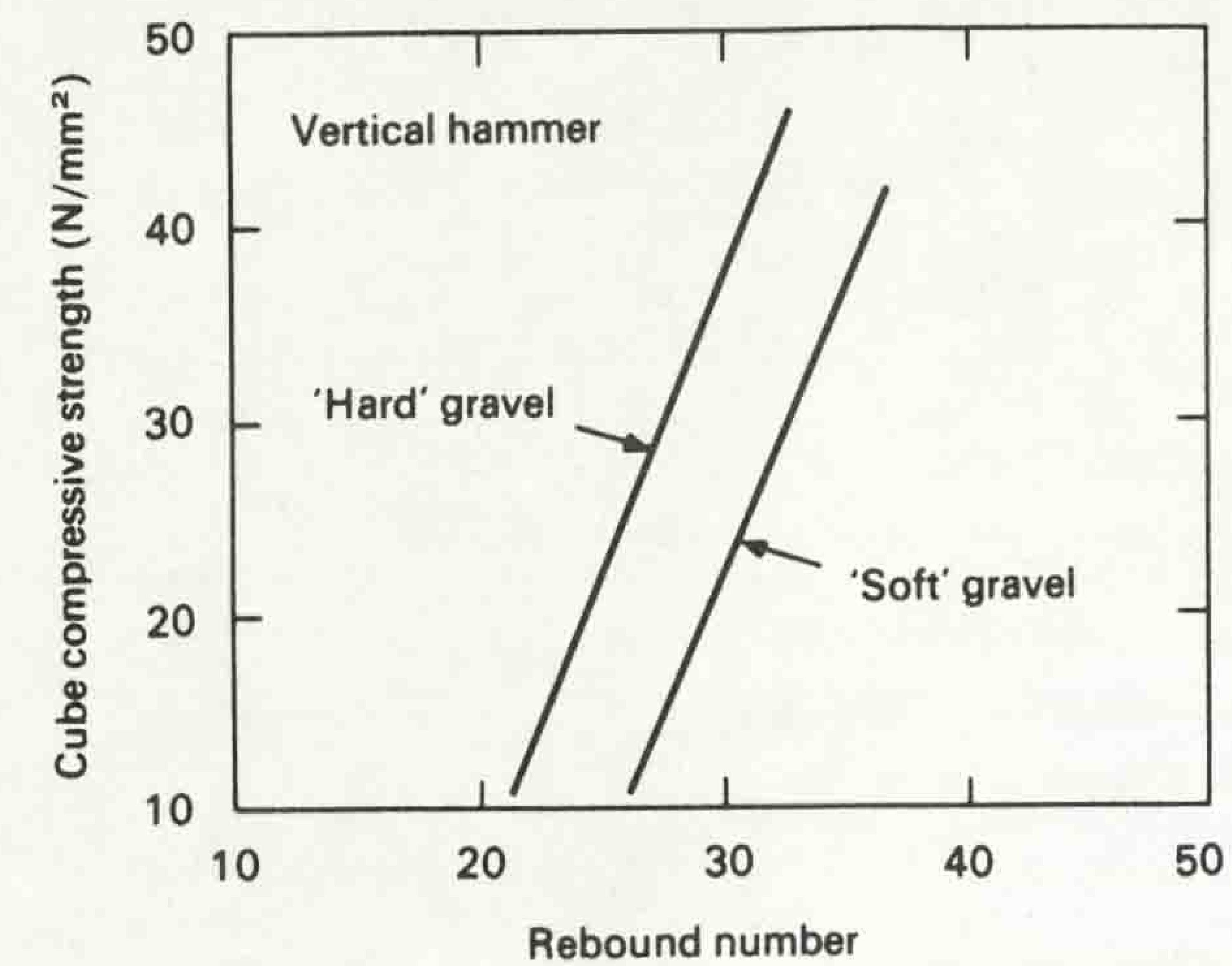

Figure 2.3 Comparison of hard and soft gravels.

depending on the source and nature, and Figure 2.3 compares typical curves for high- and low-quality gravels. These have measured hardness expressed in terms of the Mohs' Number (see section 4.1.2) of 7 and 3 respectively.

Lightweight aggregates may be expected to yield results significantly different to those for concrete made with dense aggregates, and considerable variations have also been found between various types of lightweight aggregates (22). Calibrations can however be obtained for specific lightweight aggregates, although the amount of natural sand used will affect results.

2.3.1.2 *Member characteristics*

(a) *Mass.* The effective mass of the concrete specimen or member under test must be sufficiently large to prevent vibration or movement caused by the hammer impact. Any such movement will result in a reduced rebound number. For some structural members the slenderness or mass may be such that this criterion is not fully satisfied, and in such cases absolute strength prediction may be difficult. Strength comparisons between or within individual members must also take account of this factor. The mass of calibration specimens may be effectively increased by clamping them firmly in a heavy testing machine, and this is discussed more fully in section 2.3.2.

(b) *Compaction.* Since a smooth, well-compacted surface is required for the test, variations of strength due to internal compaction differences cannot be detected with any reliability. All calibrations must assume full compaction.

(c) *Surface type.* Hardness methods are not suitable for open-textured or exposed aggregate surfaces. Trowelled or floated surfaces may be harder than moulded surfaces, and will certainly be more irregular. Whilst they may be smoothed by grinding, this is laborious and it is best to avoid trowelled surfaces in view of the likely overestimation of strength from hardness readings. The absorption and smoothness of the mould surface will also have a considerable effect. Calibration specimens will normally be cast in steel moulds which are smooth and non-absorbent, but more absorbent shuttering may well produce a harder surface, and hence strength will be overestimated. Whilst moulded surfaces are preferred for on-site testing, care must be taken to ensure that strength calibrations are based on similar surfaces, since considerable errors can result from this cause.

(d) *Age, rate of hardening and curing type.* The relationship between hardness and strength has been shown to vary as a function of time (20), and variations in initial rate of hardening, subsequent curing, and exposure conditions will further influence this relationship. Where heat treatment or some other form of accelerated curing has been used, a specific calibration will be necessary. The moisture state may also be influenced by the method of curing. For practical purposes the influence of time may be regarded as unimportant up to the age of three months, but for older concretes it may be possible to develop reduction factors which take account of the concrete's history.

(e) *Surface carbonation.* Concrete exposed to the atmosphere will normally form a hard carbonated skin, whose thickness will depend upon the exposure conditions and age. It may be up to 20 mm for old concrete although it is unlikely to be significant at ages of less than three months. The depth of carbonation can easily be determined by testing with phenolphthalein, which will remain colourless in the carbonated zone. Examination of gravel concrete specimens which had been exposed to an outdoor "city-centre" atmosphere for six months showed a carbonated depth of only 4 mm. This was not sufficient to influence the rebound number/strength relationship in comparison with similar specimens stored in a laboratory atmosphere, although for these specimens no measurable skin was detected. In extreme cases, however, it is known that the overestimate of strength from this cause may be up to 50 %, and is thus of great importance. Where carbonation is known to exist, correction factors can be obtained by removing the surface layer over a small area and comparing the results taken on this surface with those for the carbonated surface in the vicinity.

(f) *Moisture condition.* The hardness of a concrete surface is lower when wet than when dry, and the rebound vs. strength relationship will be influenced

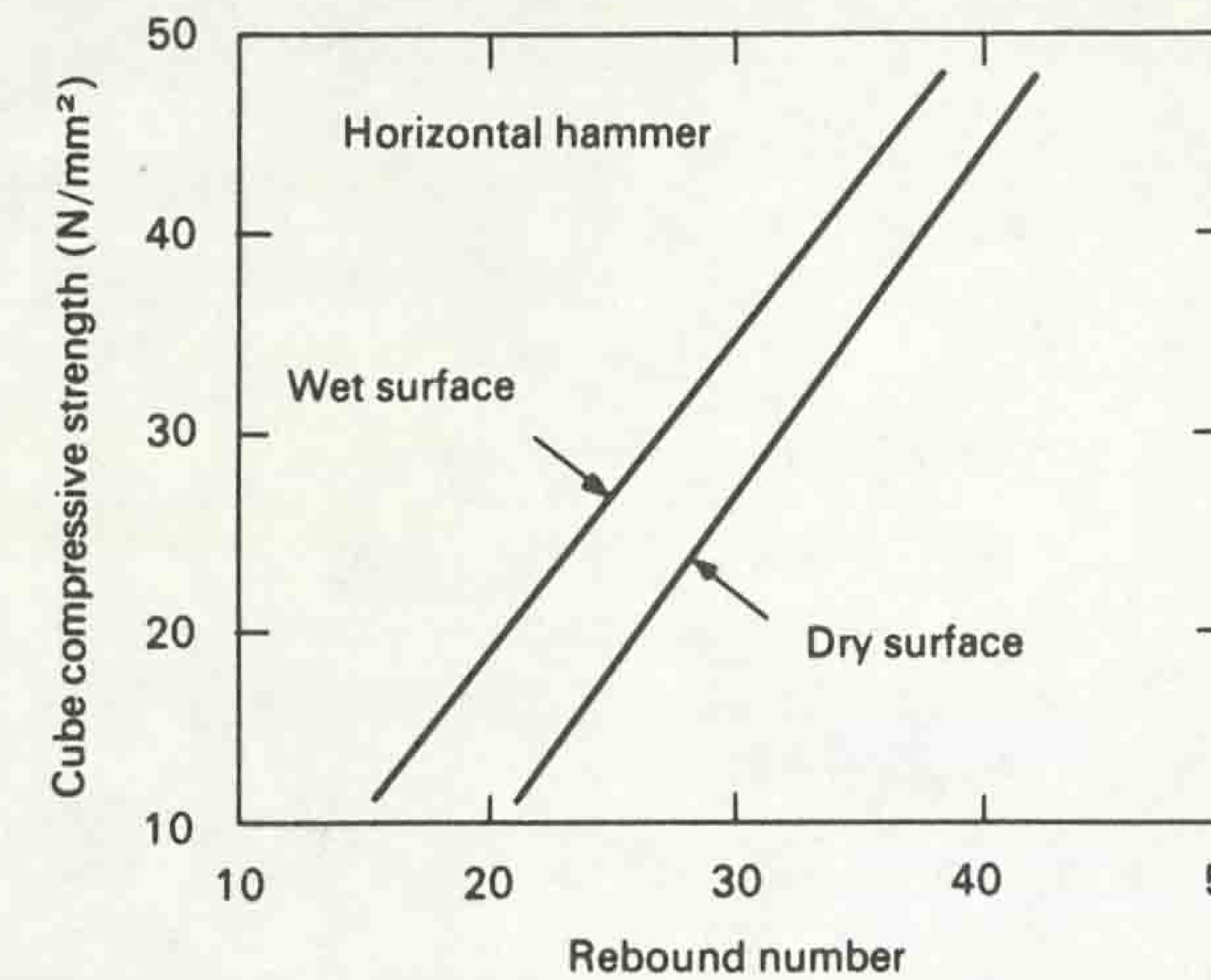

Figure 2.4 Influence of surface moisture condition (based on ref. 23).

accordingly. This effect is illustrated by Figure 2.4, based on work by the US army (23), from which it will be seen that a wet surface test may lead to an underestimate of strength of up to 20 %. Field tests and strength calibrations should normally be based on dry surface conditions, but the effect of internal moisture on the strength of control specimens must not be overlooked. This is considered in more detail in section 2.3.2.

(g) *Stress state and temperature.* Both these factors may influence hardness readings, although in normal practical situations this is likely to be small in comparison with the many other variables. Particular attention should, however, be paid to the functioning of the test hammer if it is to be used under extremes of temperature.

2.3.2 *Calibration*

Clearly the influences of the variables described above are so great that it is very unlikely that a general calibration curve relating rebound number to strength, as provided by the equipment manufacturers, will be of any practical value. Strength calibration must be based on the particular mix under investigation, and the mould surface, curing and age of laboratory specimens should correspond as closely as possible to the in-place concrete. Correct functioning of the rebound hammer must also be checked occasionally using a standard steel anvil of known mass. This is necessary because wear may change the spring and internal friction characteristics of the equipment.

Calibrations prepared for one hammer will also not necessarily apply to another.

The importance of specimen mass has been discussed above; it is essential that test specimens are either securely clamped in a heavy testing machine or supported upon an even solid floor. Cubes or cylinders of at least 150 mm should be used, and a minimum restraining load of 15% of the specimen strength has been suggested for cylinders (17) whilst BS 4408 (18) recommends not less than 7 N/mm² for cubes. Some typical relationships between rebound number and restraining load are given in Figure 2.5, which shows that once a sufficient load has been reached the rebound number remains reasonably constant.

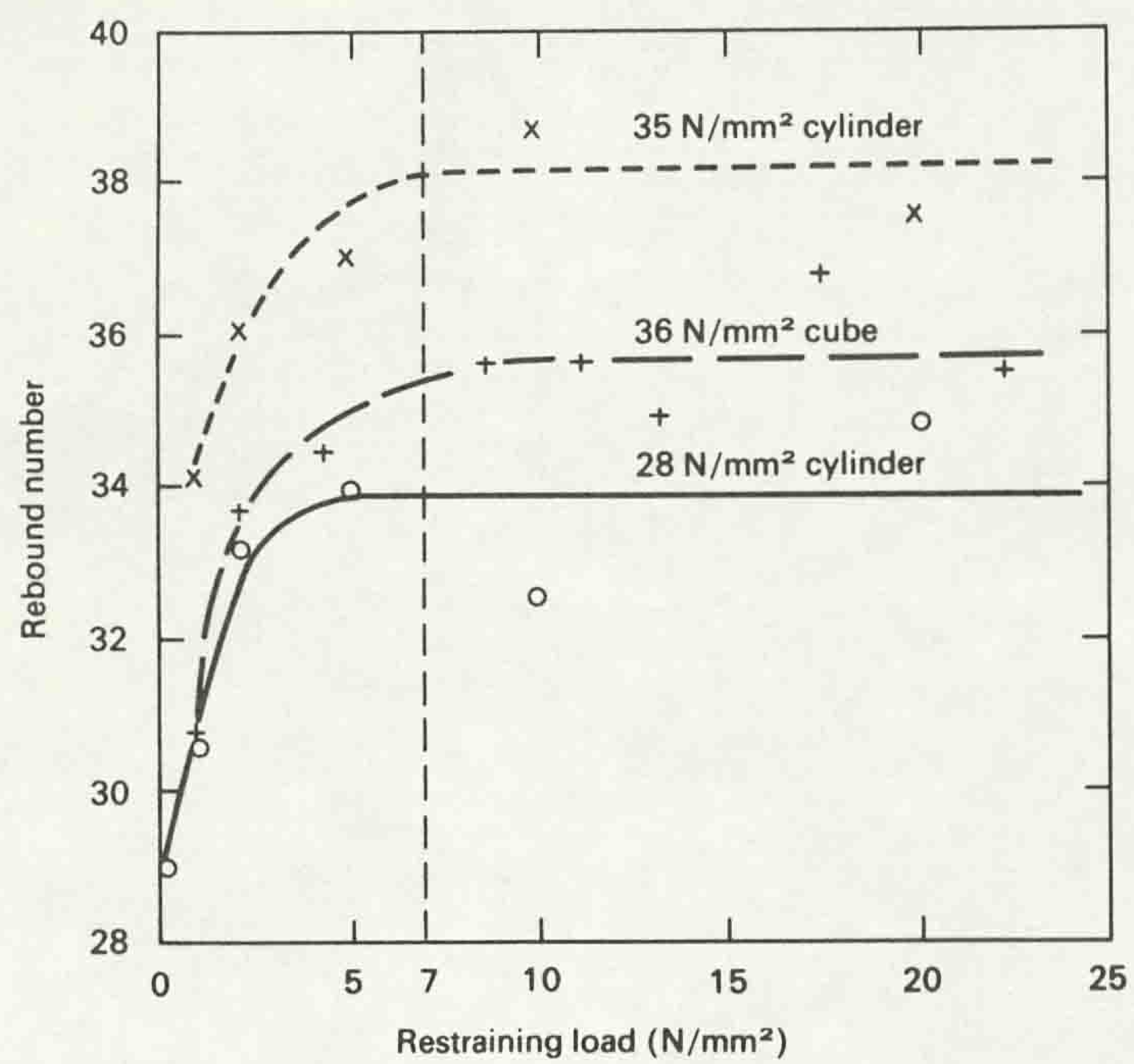

Figure 2.5 Effect of restraining load on calibration specimen (incorporating data from ref. 17).

It is well established that the strength of a cube tested wet is likely to be about 10% lower than the strength of a corresponding cube tested dry. Since rebound measurements should be taken on a dry surface, it is recommended (18) that wet cured cubes be dried in the laboratory atmosphere for 24 hours before test, and it is therefore to be expected that they will yield higher strengths than if tested wet in the standard manner (24). Depending upon the purpose of the test programme it may be necessary to confirm this

relationship, and the relative moisture conditions of the calibration specimens and in-place concrete must also be considered when interpreting the field results.

Readings should be taken on at least two vertical faces of the specimen as cast, as described in section 2.2, and the hammer orientation must be similar to that to be used for the in-place tests. The influence of gravity on the mass will depend on whether it is moving vertically up or down, horizontally or on an inclined plane. The effect on the rebound number will be considerable, although the relative values suggested by the manufacturer are likely to be reliable in this instance because this is purely a function of the equipment.

2.3.3 *Interpretation*

The interpretation of surface hardness readings relies upon a knowledge of the extent to which the factors described in section 2.3.1 have been standardized between readings being compared. This applies whether the results are being used to assess relative quality or to estimate strength. It will be apparent from Figure 2.6, which shows a typical strength calibration chart produced under "ideal" laboratory conditions, that the scatter of results is considerable, and the strength range corresponding to a given rebound number is about ±15% even for "identical" concrete. In any practical situation it is very unlikely that a strength prediction can be made to an accuracy better than ±25% (17, 18). The calibration scatter also suggests that even if a strength prediction is not required, a considerable variation of rebound number can be expected for "identical" concrete, and acceptable

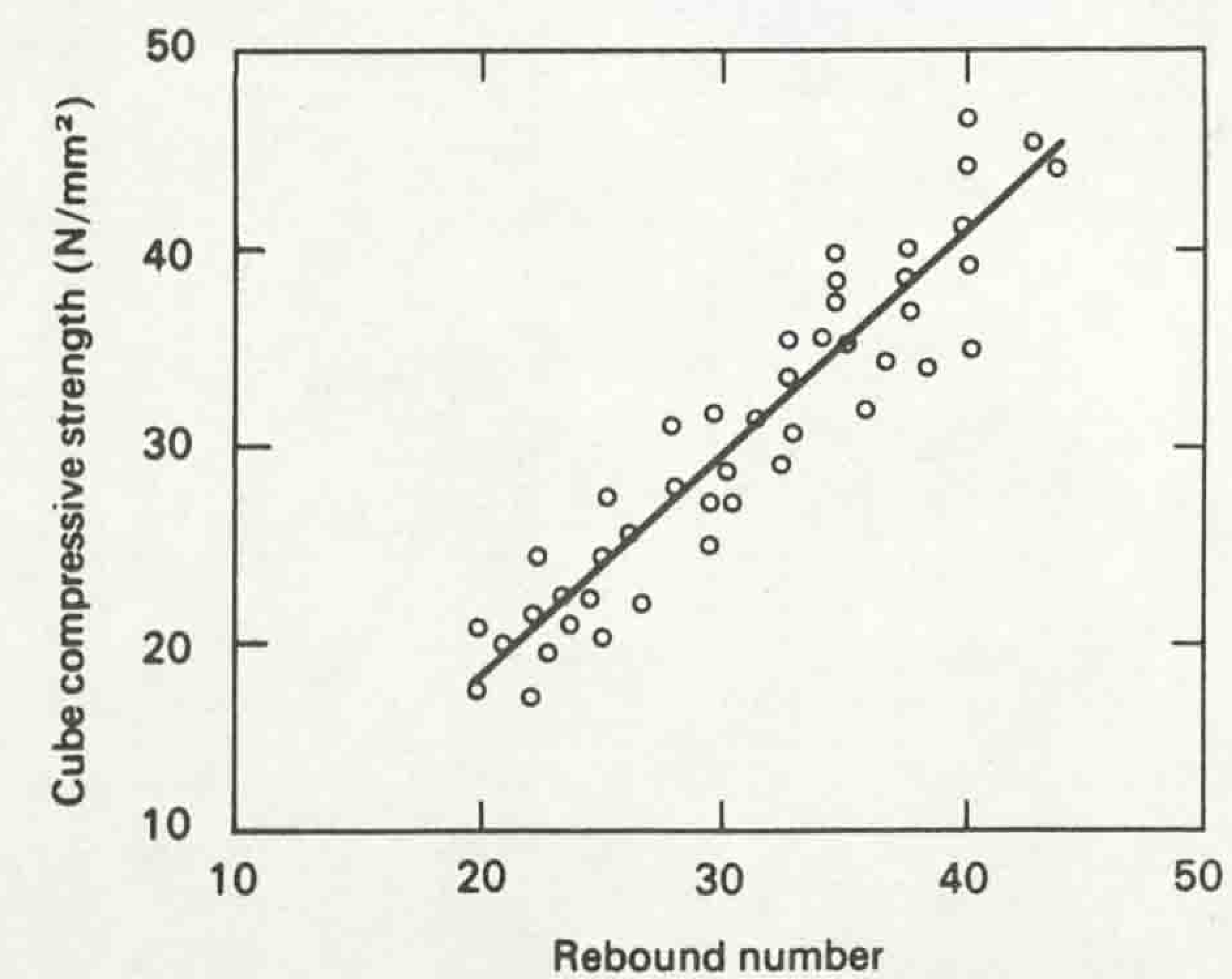

Figure 2.6 Typical rebound number/compressive strength calibration chart.

limits must be determined in conjunction with some other form of testing. It is suggested (10) that where the total number of readings (n) taken at a location is not less than 10, the accuracy of the mean rebound number is likely to be within $\pm 15/\sqrt{n}\%$ with 95% confidence. The results may usefully be presented in graphical form as described in section 1.5.2.1, and calculation of the coefficient of variation may yield an indication of concrete uniformity, as described in section 1.5.2.2, when sufficient results are available.

The test location within the member is important when interpreting results (Chapter 1) but it should be noted that the test yields information about a thin surface layer only. Results are unrelated to the properties of the interior, and furthermore are not regarded as reliable on concrete over three months old unless special steps are taken to allow for age effects and surface carbonation, as described above.

Whilst it is generally the relationship between rebound number and compressive strength that is of interest, it has been shown (21, 22) that similar relationships can be established with flexural strength although with an even greater scatter. It appears (17) that no general relationship between rebound number and elastic modulus exists although it may be possible to produce such a calibration for a specific mix.

2.3.4 *Applications and limitations*

The useful applications of surface hardness measurements can be divided into four categories

(a) checking the uniformity of concrete quality
(b) comparing a given concrete with a specified requirement
(c) approximate estimation of strength
(d) abrasion resistance classification.

Whatever the application, it is essential that the factors influencing test results are standardized or allowed for, and it should be remembered that results relate only to the surface zone of the concrete under test. A further overriding limitation relates to testing at early ages or low strengths, because the rebound numbers may be too low for accurate reading and the impact may also cause damage to the surface (Figure 2.7). It is therefore not recommended that the method is used for concrete which has a cube strength of less than 10 N/mm^2 or which is less than 7 days old, unless of high strength.

(a) *Concrete uniformity checking.* The most important and reliable applications of surface hardness testing are where it is not necessary to attempt to convert the results to some other property of the concrete. It is claimed (20) that surface hardness measurements give more consistently

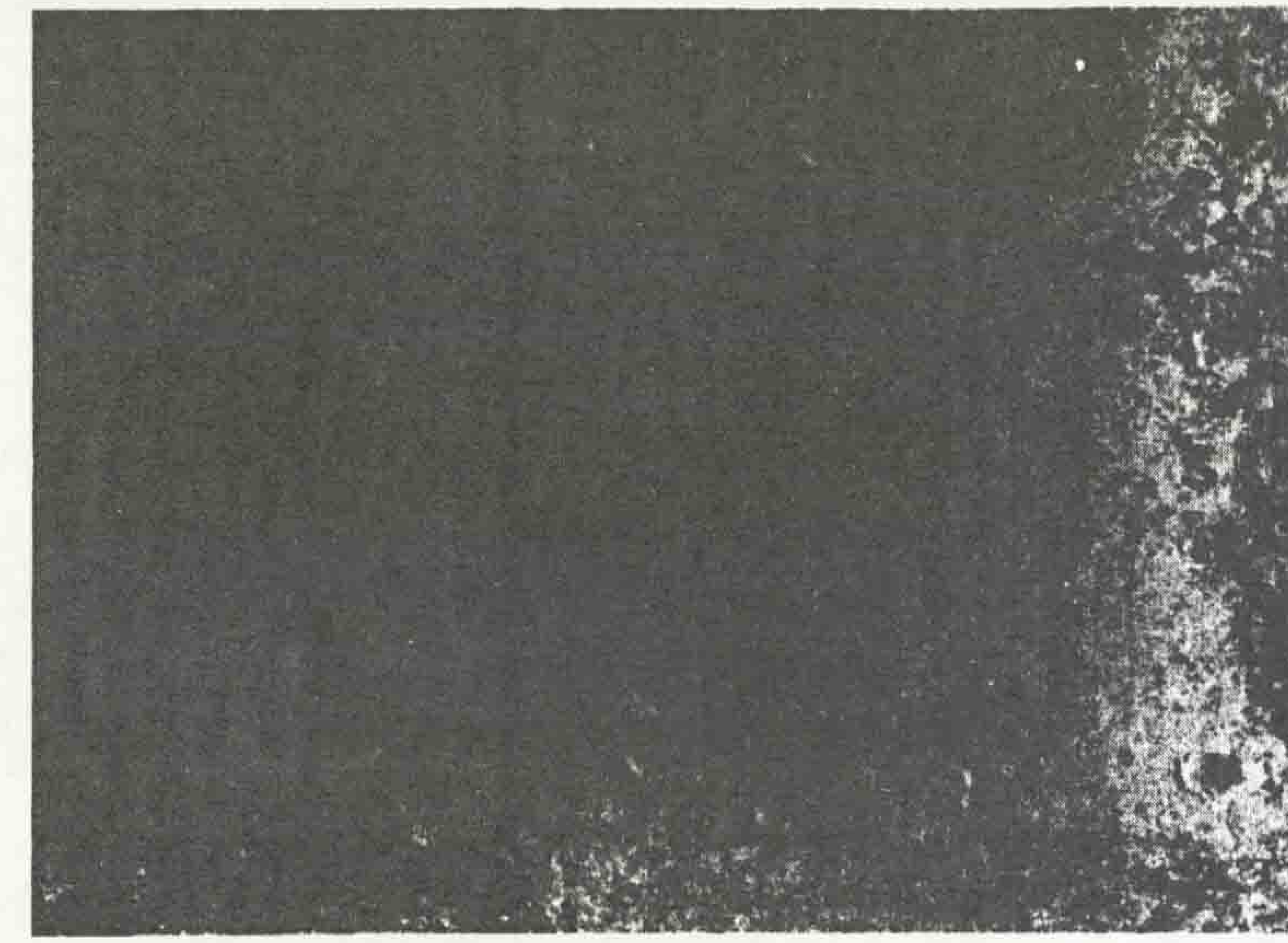

Figure 2.7 Surface damage on green concrete.

reproducible results than any other method of testing concrete. Although they do not detect poor internal compaction, results are sensitive to variations of quality between batches, or due to inadequate mixing or segregation. The value as a control test is further enhanced by the ability to monitor the concrete in members cheaply and more comprehensively than is possible by a small number of control specimens. For such comparisons to be valid for a given mix it is only necessary to standardize age, maturity, surface moisture conditions (which should preferably be dry), and location on the structure or unit.

This approach has been extensively used to control uniformity of precast concrete units, and may also prove valuable for the comparison of suspect in-situ elements with similar elements which are known to be sound. A further valuable use for such comparative tests may be to establish the representation of other forms of testing, possibly destructive, which may yield more specific but localized indications of quality.

(b) *Comparison with a specific requirement.* This application is also popular in the precasting industry, where a minimum hardness reading may be calibrated against some specific requirement of the concrete. For instance, the readiness of precast units for transport may be checked, with calibration based on proof load tests. The approach may also be used as an acceptance criterion, in relation to the removal of temporary supports from structural members, or in commencement of stress transfer in prestressed concrete construction.

(c) *Approximate strength estimation.* This represents the least reliable application and (unfortunately, since a strength estimate is frequently required by engineers) is where misuse is most common. The accuracy depends entirely upon the elimination of influences which are not taken into account in the calibration. For laboratory specimens cast, cured and tested under conditions identical to those used for calibration, it is unlikely that a strength estimate better than $\pm 15\%$ can be achieved for concrete up to three months old. Whilst it may be possible to correct for one or two variables, the accuracy of absolute strength prediction will decline and is unlikely to be better than $\pm 25\%$. The use of the rebound hammer for strength estimation of in-place concrete must never be attempted unless specific calibration charts are available, and even then, the use of this method alone is not recommended, although the value of results may be improved if used in conjunction with other forms of testing as described in Chapter 1.

(d) *Abrasion resistance classification.* Abrasion resistance is generally affected by the same influences as surface hardness, and Chaplin (25) has recently suggested that the rebound hammer may be used to classify this property. This is discussed in Chapter 7.

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Paper 4

"Ultrasonic Methods"

The Testing of Concrete in Structures

Chapter 3 Surrey University Press

1982

3 Ultrasonic methods

The first reports of the measurement of the velocity of mechanically generated pulses through concrete appeared in the USA in the mid 1940's. It was found that the velocity depended primarily upon the elastic properties of the material and was almost independent of geometry. The potential value of this approach was apparent, but measurement problems were considerable, and led to the development in France, a few years later, of repetitive mechanical pulse equipment. At about the same time work was undertaken in Canada and the United Kingdom using electro-acoustic transducers, which were found to offer greater control on the type and frequency of pulses generated. This form of testing has been developed into the modern ultrasonic method, employing pulses in the frequency range of 50–150 per second, generated and recorded by electronic circuits. The method has become widely accepted around the world and commercially-produced robust lightweight equipment suitable for site as well as laboratory use is readily available.

If properly used by an experienced operator, a considerable amount of information about the interior of a concrete member can be obtained. However, since the range of pulse velocities relating to practical concrete strengths is relatively small, great care is necessary, especially for site usage. Furthermore, since it is the elastic properties of the concrete which affect pulse velocity, it is often necessary to consider in detail the relationship between elastic modulus and strength when interpreting results. Recommendations for the use of this method are given in BS 4408 pt. 5 (26) and also in ASTM C597-71 (27).

3.1 Theory of pulse propagation through concrete

Three types of waves are generated by an impulse applied to a solid mass. Surface waves having an elliptical particle displacement are the slowest, whilst shear or transverse waves with particle displacement at right angles to the direction of travel are faster. Longitudinal waves with particle displacement in the direction of travel (sometimes known as compression waves) are the most important since these are the fastest and provide more useful information. Electro-acoustical transducers produce waves primarily of this type; other types generally cause little interference because of their lower speed.

The wave velocity depends upon the elastic properties and mass of the medium, and hence if the mass and velocity of wave propagation are known it is possible to assess the elastic properties. For an infinite, homogeneous, isotropic elastic medium, the compression wave velocity is given by:

$$V = \sqrt{\frac{K \cdot E_d}{\rho}}$$

where V = compression wave velocity (km/sec)

$$K = \frac{(1-\nu)}{(1+\nu)(1-2\nu)}$$

E_d = dynamic modulus of elasticity (kN/mm^2)

ρ = density (kg/m^3)

and ν = dynamic Poisson's ratio.

In this expression the value of K is relatively insensitive to variations of the dynamic Poisson's ratio ν, and hence provided that a reasonable estimate of this value and the density can be made, it is possible to compute E_d using a measured value of wave velocity V. Since ν and ρ will vary little for mixes with natural aggregates, the relationship between velocity and dynamic elastic modulus may be expected to be reasonably consistent despite the fact that concrete is not necessarily the "ideal" medium to which the mathematical relationship applies, as indicated in section 3.3.

3.2 Equipment and use

3.2.1 *Equipment*

The test equipment must provide a means of generating a pulse, transmitting this to the concrete, receiving and amplifying the pulse and measuring and displaying the time taken. The basic circuitry requirements are shown in Figure 3.1.

Repetitive voltage pulses are generated electronically and transformed into wave bursts of mechanical energy by the transmitting transducer, which must be coupled to the concrete surface through a suitable medium (see section 3.2.2). A similar receiving transducer is also coupled to the concrete at a known distance from the transmitter, and the mechanical energy converted

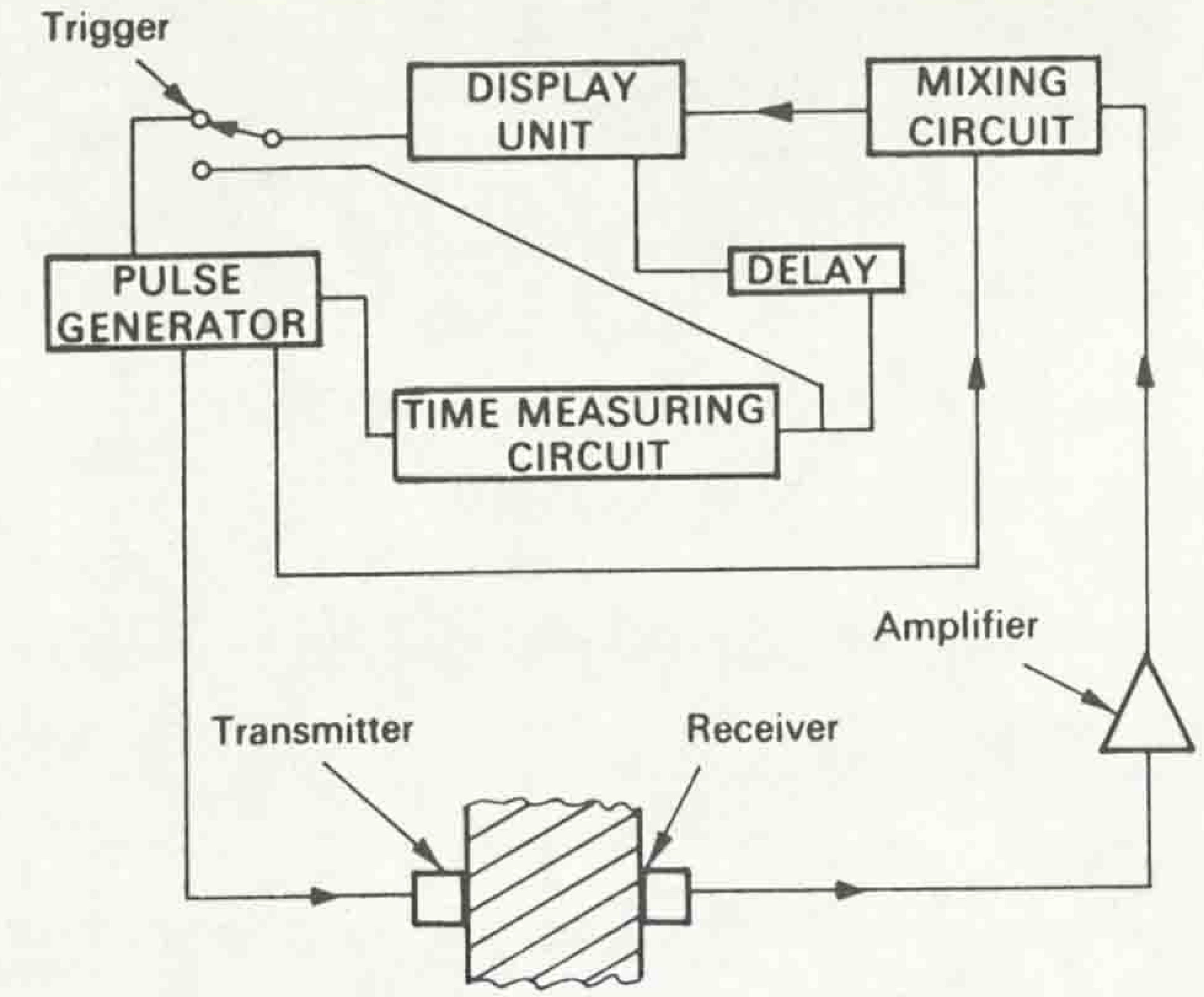


Figure 3.1 Typical U.P.V. testing equipment.

back to electrical pulses of the same frequency. The electronic timing device measures the interval between the onset and reception of the pulse and this is displayed either on an oscilloscope or as a digital readout. The equipment must be able to measure the transit time to an accuracy of $\pm 1\%$. To ensure a sharp pulse onset, the electronic pulse to the transmitter must have a rise time of less than one-quarter of its natural period. The repetition frequency of the pulse must be low enough to avoid interference between consecutive pulses, and the performance must be maintained over a reasonable range of climatic and operating conditions.

Transducers with natural frequencies between 20 kHz and 150 kHz are the most suitable for use with concrete, and these may be of any type, although the piezo-electric crystal is most popular. Time measurement is based on detection of the compressive wave pulse, the first part of which may have only a very small amplitude. If an oscilloscope is used, the received pulse is amplified and the onset taken as the tangent point between the signal curve and the horizontal time-base line, whilst for digital instruments the pulse is amplified and shaped to trigger the timer from a point on the leading edge of the pulse.

A number of commercially-produced instruments have become available in recent years which satisfy these requirements. The most popular of these are the V-meter produced in the USA (28) and the PUNDIT (Portable Ultrasonic Non-Destructive Digital Indicating Tester) (29) produced in the United Kingdom. These have many similarities: both measure 180 × 110 × 160 mm,

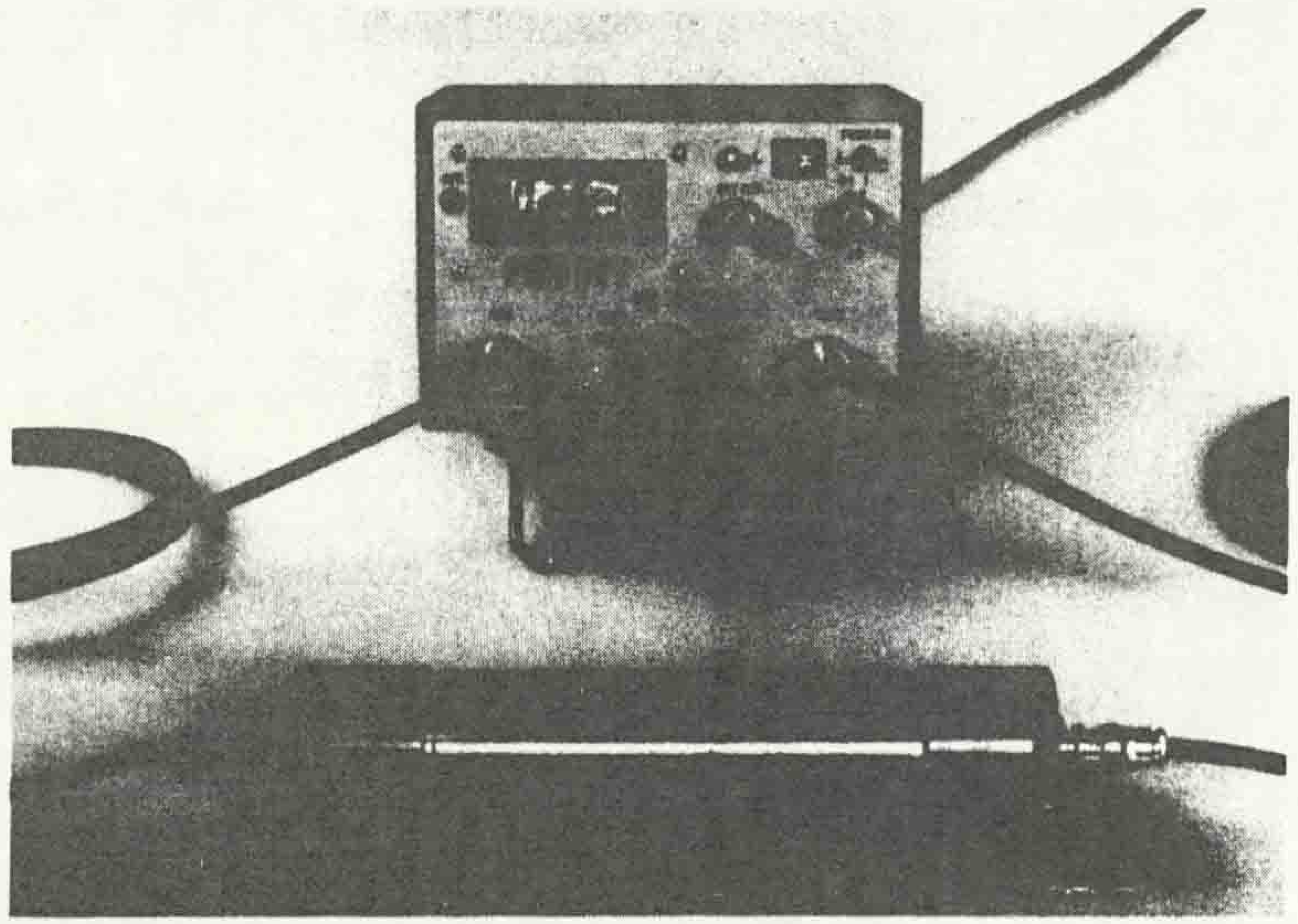


Figure 3.2 PUNDIT in laboratory (photograph by courtesy of C.N.S. Instruments Ltd.).

weigh 3.2 kg, and have a digital display. Nickel-cadmium rechargeable batteries allow over five hours continuous operation. Both also incorporate constant current chargers to enable recharging from an a.c. mains supply, and may also be operated directly from the mains through a mains supply unit. For use in the laboratory an analogue unit can be added and this in turn can be connected to a recorder for continual experimental monitoring.

Figure 3.2 shows the PUNDIT set up in the laboratory with 54 kHz transducers and a calibration reference bar. This steel bar has known characteristics and is used to set the zero of the instrument by means of a variable delay control unit each time that it is used. The display is a four-digit liquid crystal and gives a direct transit time reading in microseconds. A wide range of transducers between 24 kHz and 200 kHz is available, although the 54 kHz and 82 kHz versions will normally be used for site or laboratory testing of concrete. Waterproof and even deep-sea versions of these transducers are available. An alternative form is the exponential probe transducer which makes a point contact, and offers operating advantages over flat transducers on rough or curved surfaces. The power output by such a transmitter is small and of little value in concrete testing, but the receiver (Figure 3.3) is very useful. The equipment is robust and is provided with a carrying case for site use. Signal amplifiers are also available where long path lengths are involved on site, and the range of acceptable ambient temperatures of 0–45°C should cover most practical situations.

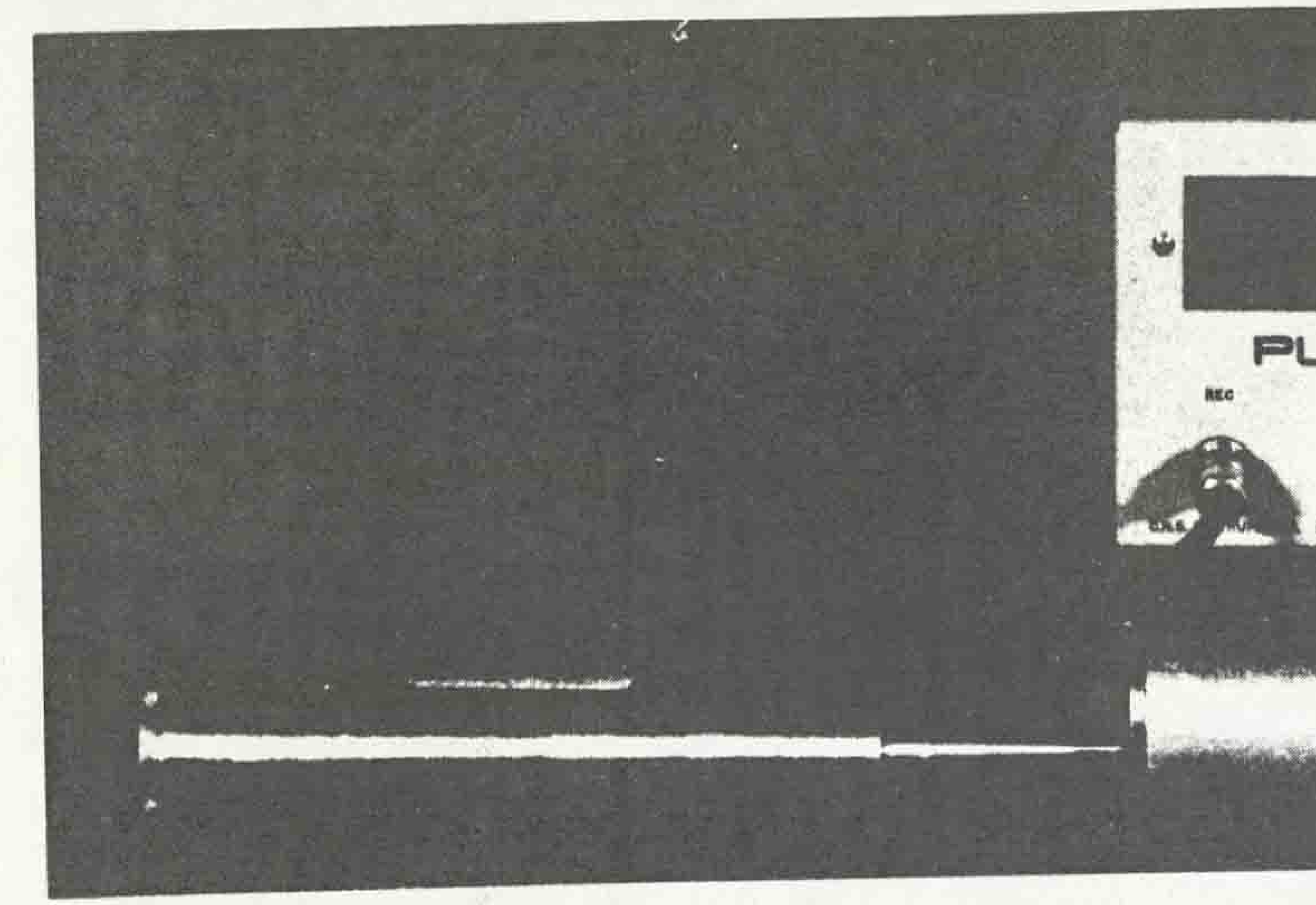


Figure 3.3 Exponential probe transducer.

3.2.2 *Use*

Operation is relatively straightforward but requires great care if reliable results are to be obtained. One essential is good acoustical coupling between the concrete surface and the face of the transducer, and this is provided by a medium such as petroleum jelly, liquid soap or grease. Air pockets must be eliminated, and it is important that only a thin separating layer exists—any surplus must be squeezed out. A light medium, such as petroleum jelly or liquid soap, has been found to be best for smooth surfaces, but a thicker grease is recommended for rougher surfaces which have not been cast against smooth shutters. If the surface is very rough or uneven, grinding or preparation with plaster of Paris or quick-setting mortar may be necessary to provide a smooth surface for transducer application. It is also important that readings are repeated by complete removal and reapplication of transducers to obtain a minimum value for the transit time. Whilst the measuring equipment is claimed to be accurate to ±0.1 microseconds, if a transit time accuracy of ±1% is to be achieved it may typically be necessary to obtain a reading to ±0.7 microseconds over a 300 mm path length. This can only be achieved with careful attention to measurement technique, and any dubious readings should be repeated as necessary, with special attention to the elimination of any other source of vibration, however slight, during the test.

The path length must also be measured to an accuracy of ±1%. This should present little difficulty with paths over about 500 mm, but for shorter lengths it is recommended that calipers be used. The nominal member dimensions shown on drawings will seldom be adequate.

3.2.2.1 Transducer arrangement. There are three basic ways in which the transducers may be arranged, as shown in Figure 3.4. These are:

(a) opposite faces (direct transmission)
(b) adjacent faces (semi-direct transmission)
(c) same face (indirect transmission).

Since the maximum pulse energy is transmitted at right angles to the face of the transmitter, the *direct* method is the most reliable from the point of view of transit time measurement. Also, the path is clearly defined and can be measured accurately, and this approach should be used wherever possible for assessing concrete quality. The *semi-direct* method can sometimes be used satisfactorily if the angle between the transducers is not too great, and if the path length is not large. The sensitivity will be smaller, and if these requirements are not met it is possible that no clear signal will be received because of attenuation of the transmitted pulse. The path length is also less clearly defined due to the finite transducer size, but it is generally regarded as adequate to take this from centre to centre of transducer faces.

The *indirect* method is definitely the least satisfactory, since the received signal amplitude may be less than 3% of that for a comparable direct transmission. The received signal is dependent upon scattering of the pulse by

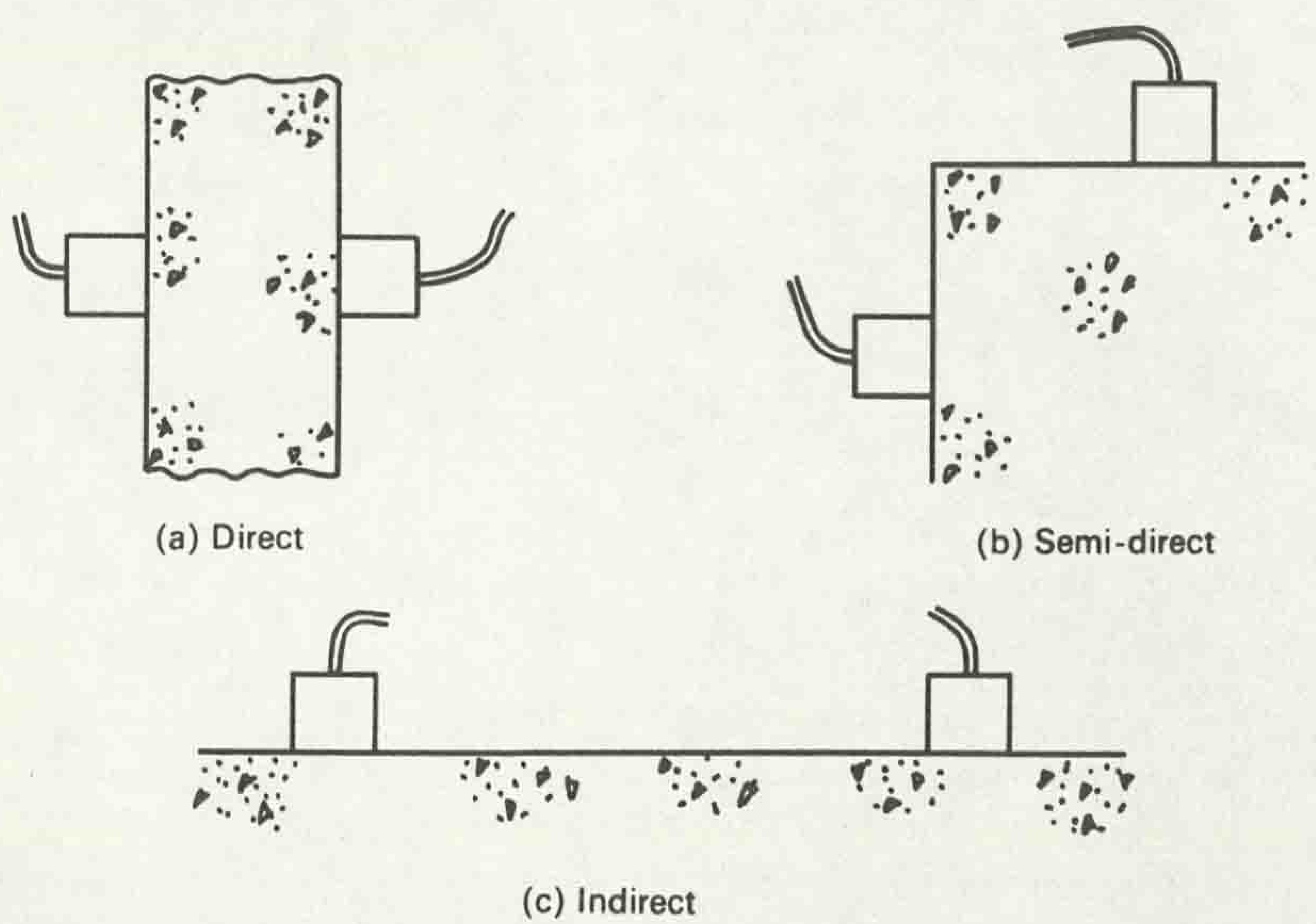


Figure 3.4 Types of reading.

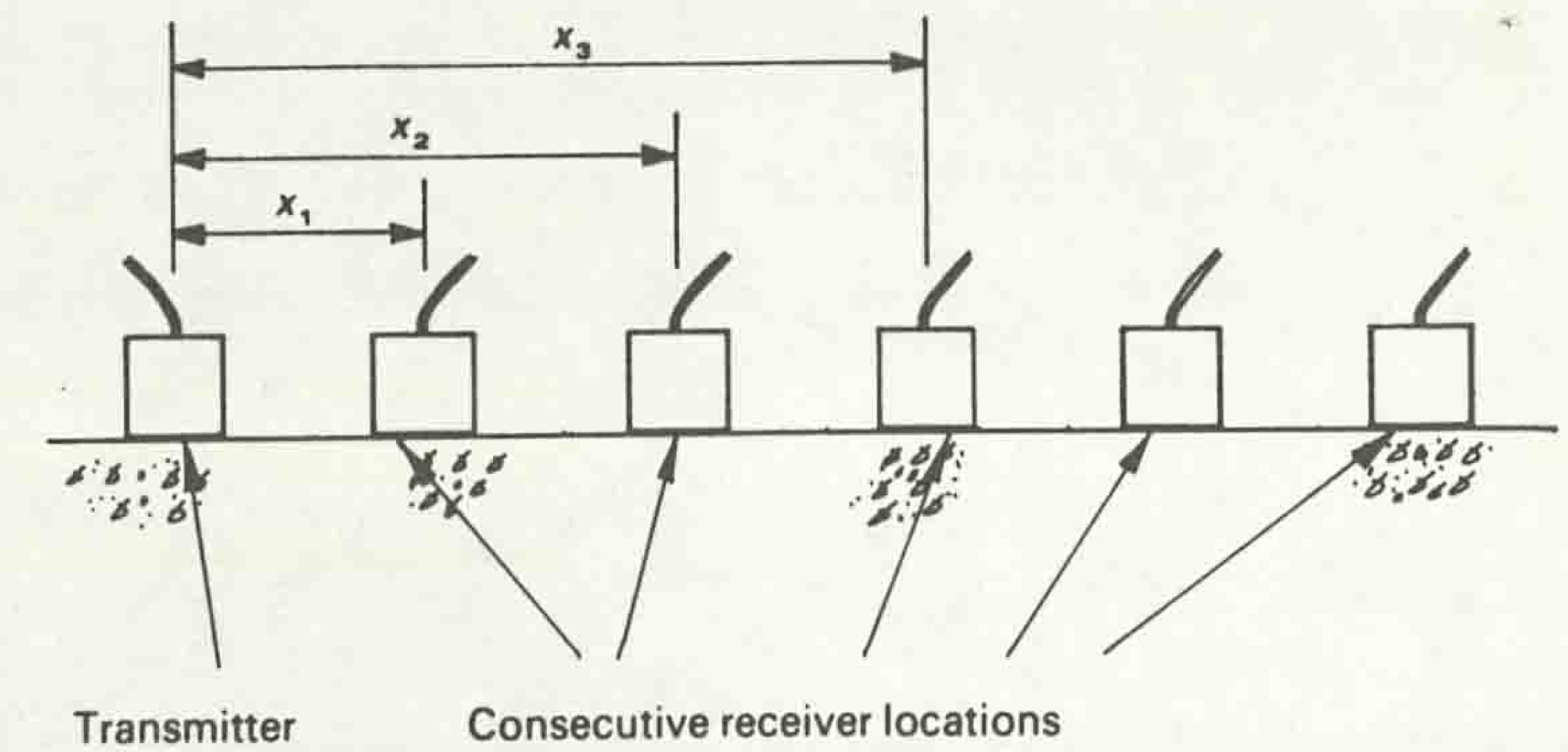


Figure 3.5 Indirect reading—transducer arrangement.

discontinuities and is thus highly subject to errors. The pulse velocity will be predominantly influenced by the surface zone concrete, which may not be representative of the body, and the exact path length is uncertain. A special procedure is necessary to account for this lack of precision of path length, requiring a series of readings with the transmitter fixed whilst the receiver is located at a series of fixed incremental points along a chosen radial line (Figure 3.5). The results are plotted (Figure 3.6) and the mean pulse velocity is given by the slope of the best straight line. If there is a discontinuity in this plot it is likely that either surface cracking or an inferior surface layer is

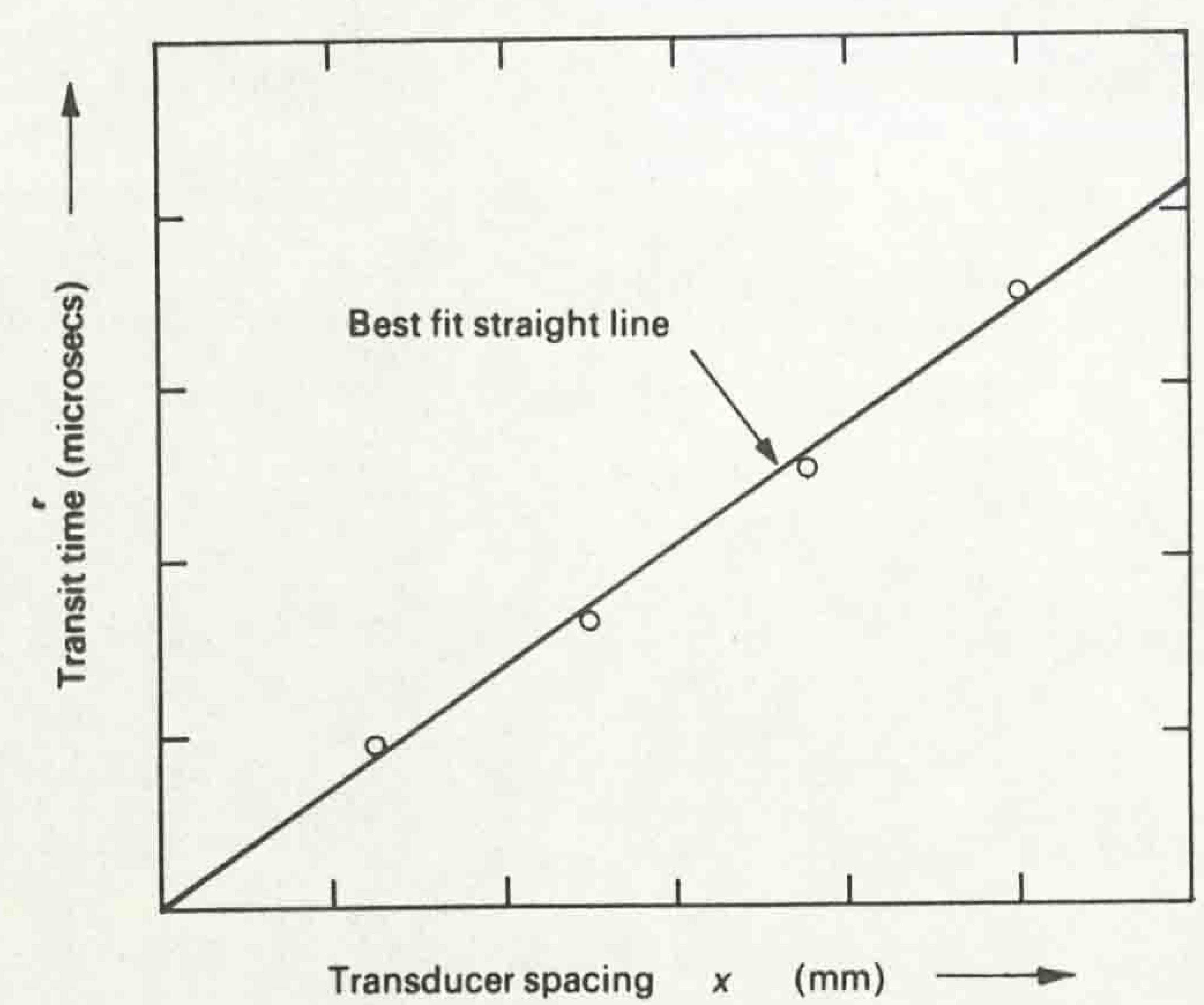


Figure 3.6 Indirect reading—results plot.

present (see section 3.4). Unless measurements are being taken to detect such features, this method should be avoided if at all possible and only used where just one surface is available.

3.2.2.2 Transducer selection. The most commonly used transducers have a natural frequency of 54 kHz (Figure 3.2). They have a flat surface of 50 mm diameter, and thus good contact must be ensured over a considerable area. However, the use of a probe transducer making only point contact and normally requiring no surface treatment or couplant offers advantages. Time savings may be considerable and path length accuracy for indirect readings may be increased, but this type of transducer is unfortunately more sensitive to operator pressure. Receivers (as shown in Figure 3.3) have been found to operate satisfactorily in the field, but the signal power available from a transmitting transducer of this type is so low that its use is not normally practicable for site testing. The exponential probe receiver, which has a tip diameter of only 6 mm may also be useful on very rough surfaces where preparatory work may otherwise be necessary.

The only important factors which are likely to require the selection of an alternative transducer frequency relate to the dimensions of the member under test. Difficulties arise with small members as the medium under test cannot be considered as effectively infinite. This will occur when the path width is less than the wavelength λ. Since λ = pulse velocity/frequency of vibration, it follows that the least lateral dimensions given in Table 3.1 should

Table 3.1 Minimum lateral path and maximum aggregate dimensions

Transducer frequency (kHz)	*Minimum lateral path dimension or max. aggregate size (mm)*	
	$V_c = 3.8\ km/sec$	$V_c = 4.6\ km/sec$
54	70	85
82	46	56
150	25	30

be satisfied. Aggregate size should similarly be less than λ to avoid reduction of wave energy and possible loss of signal at the receiver, although this will not normally be a problem. Thus for practical purposes, if the minimum member width is less than 100 mm, a higher frequency should be used. Although this may reduce the maximum acceptable path length (10 m for 54 kHz to 3 m for 82 kHz), the lower energy output associated with the higher frequency means that the problem can easily be overcome by the use of an inexpensive signal amplifier.

3.2.2.3 Equipment calibration. The time delay adjustment must be used to set the zero reading for the equipment before use, and this should also be regularly checked during and at the end of each period of use. Individual transducer and connecting lead characteristics will affect this adjustment, which is performed with the aid of a calibrated steel reference bar which has a transit time of around 25 μs. A reading through this bar (Figure 3.2) is taken in the normal way ensuring that only a very thin layer of couplant separates the bar and transducers. It is also recommended that the accuracy of transit time measurement of the equipment is checked by measurement on a second reference specimen, preferably with a transit time of around 100 μs.

3.3 Test calibration and interpretation of results

The basic problem is that the material under test consists of two separate constituents, matrix and aggregate, which have different elastic and strength properties. The relationship between pulse velocity and dynamic elastic modulus of the composite material measured by resonance tests on prisms is fairly reliable, as shown in Figure 3.7. Whilst this relationship is influenced by the value of dynamic Poisson's ratio, it has been suggested (30) that for most practical concretes the estimate of modulus of elasticity should be accurate within 10%.

3.3.1 *Strength calibration*

The relationship between elastic modulus and strength of the composite material cannot be defined simply by consideration of the properties and

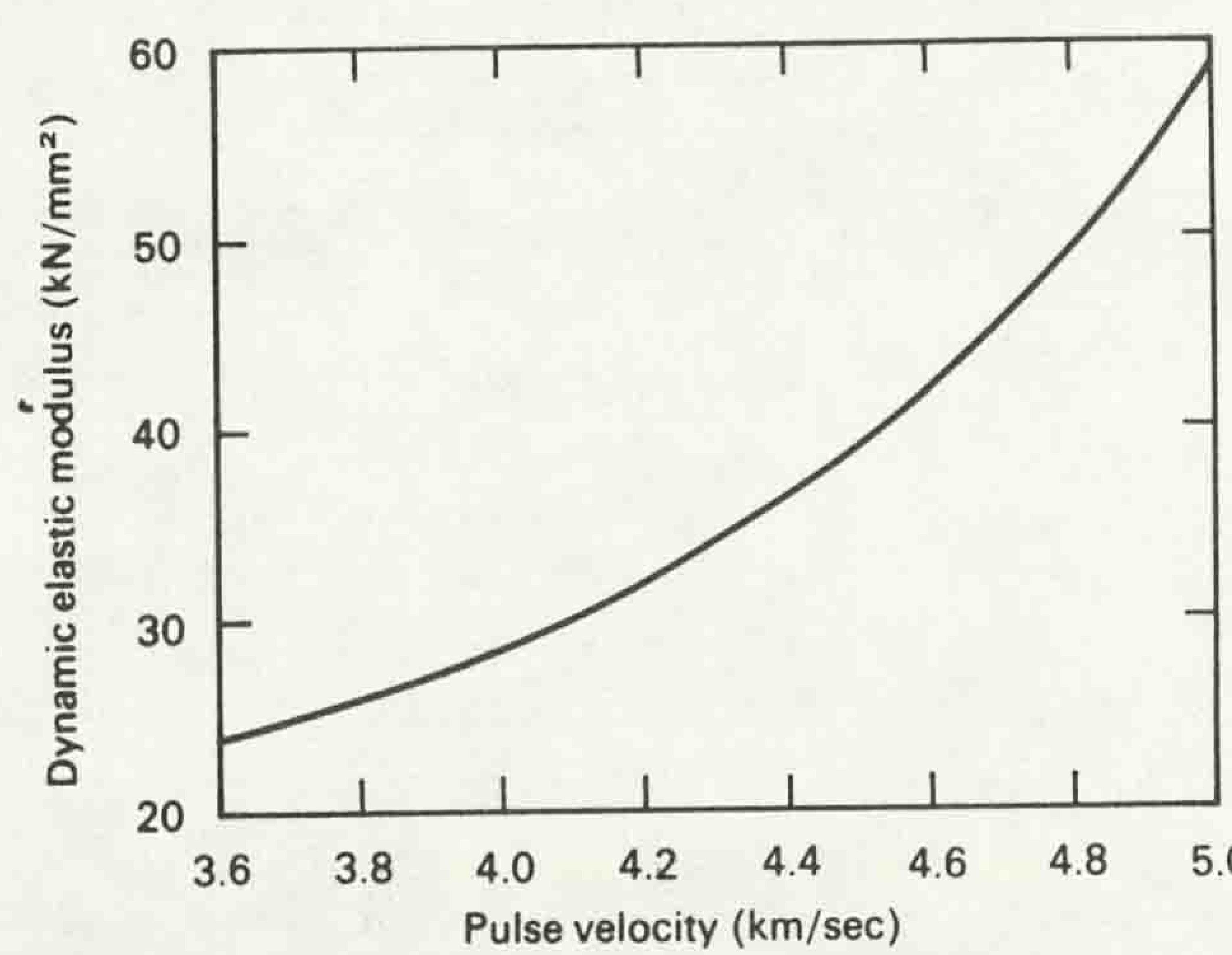


Figure 3.7 Pulse velocity vs. dynamic elastic modulus (based on ref. 26).

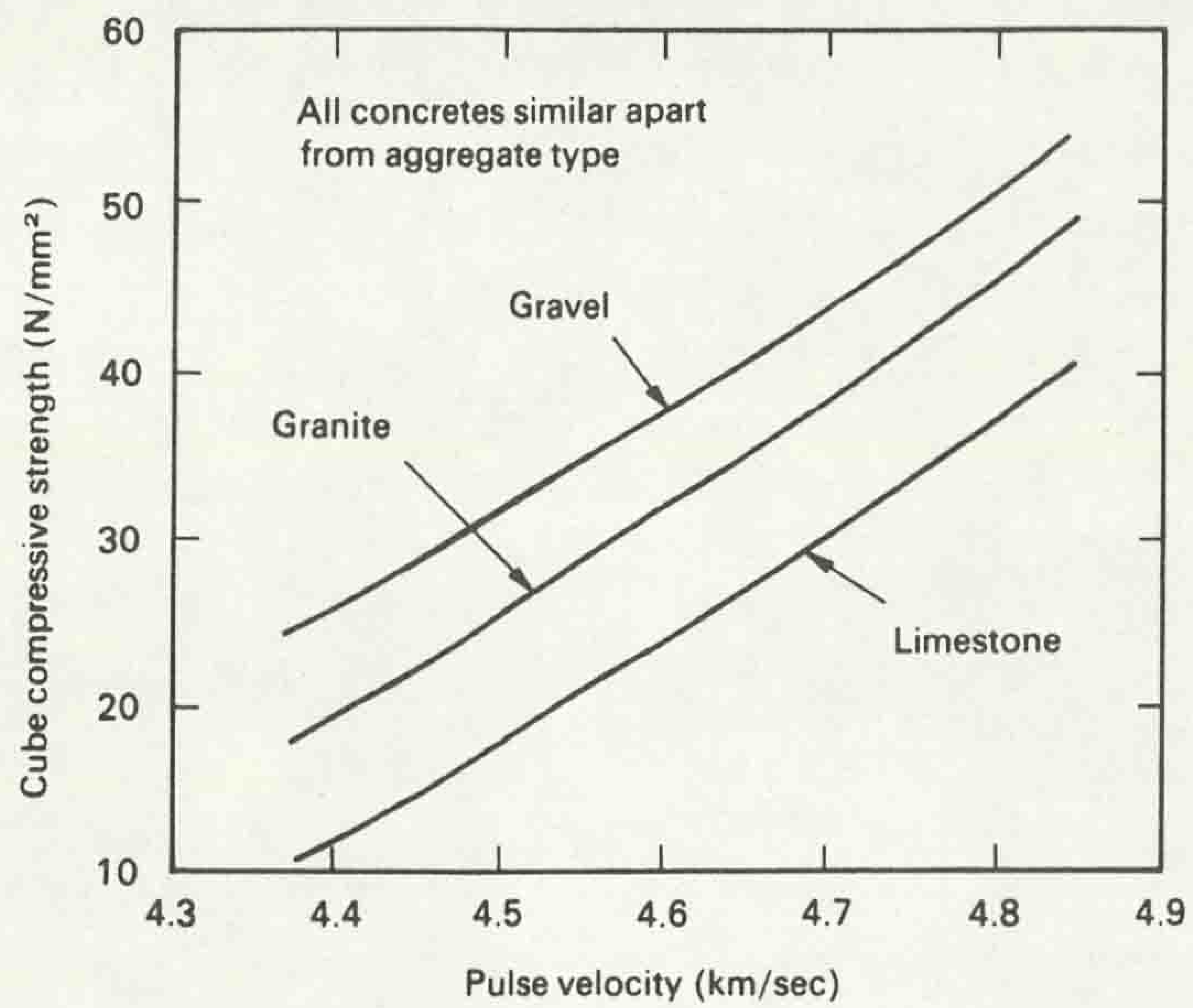


Figure 3.8 Effect of aggregate type (based on ref. 31).

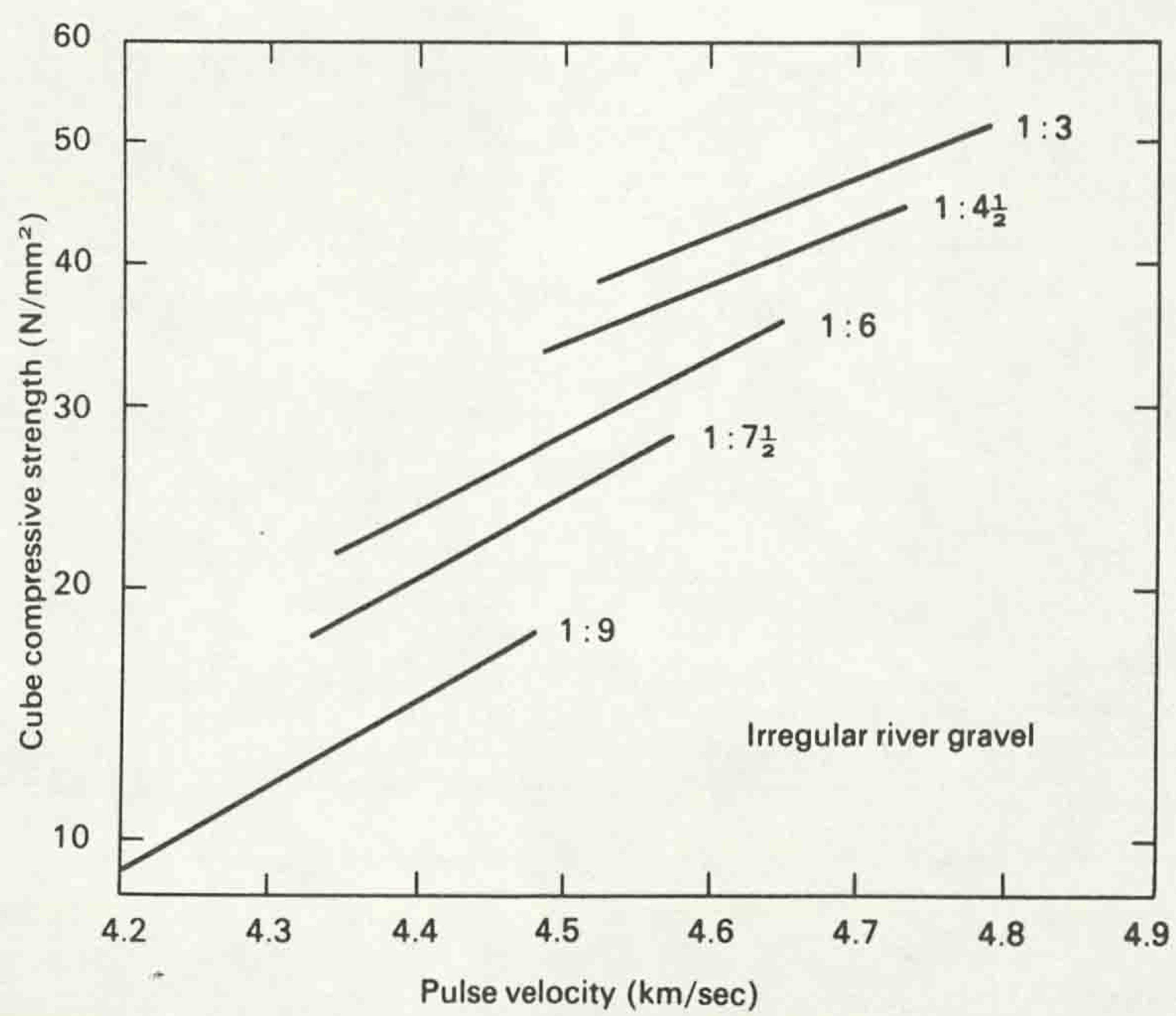


Figure 3.9 Effect of aggregate proportions (based on ref. 31).

proportions of individual constituents. This is because of the influence of aggregate particle shape, efficiency of the aggregate/matrix interface and variability of particle distribution, coupled with changes of matrix properties with age. Although some attempts have been made to represent this theoretically, the complexity of the interrelationships is such that experimental calibration for elastic modulus and pulse velocity/strength relationships is normally necessary. Aggregate may vary in type, shape, size and quantity, while the cement type, sand type, water/cement ratio and maturity are all important factors which influence the matrix properties and hence strength correlations. A pulse velocity/strength curve obtained with maturity as the only variable, for example, will differ from that obtained by varying the water/cement ratio for otherwise similar mixes, but testing at comparable maturities. Similarly, separate correlations will exist for varying aggregate types and proportions (Figures 3.8 and 3.9) as well as for cement characteristics.

Strength calibration for a particular mix should normally be undertaken in the laboratory with due attention to the factors listed above. Pulse velocity readings are taken between both pairs of opposite cast faces of cubes of known moisture condition, which are then crushed in the usual way. Ideally, at least 10 sets of three specimens should be used, covering as wide a range of strength as possible, with the results of each group averaged. A minimum of three pulse velocity measurements should be taken for each cube, and each individual reading should be within 5% of the mean for that cube. Where this is not possible, cores cut from the hardened concrete may sometimes be used for calibration, although there is a danger that drilling damage may affect pulse velocity readings. Wherever possible readings should be taken at core locations prior to cutting. Provided that cores are greater than 100 mm in diameter, and that the ends are suitably prepared prior to test, it should be possible to obtain a good calibration, although this will usually cover only a restricted strength range. If it is necessary to use smaller diameter cores, high frequency transducers (section 3.2.2.2) may have to be used, and the accuracy of crushing strength will also be reduced (see 5.3).

Although the precise relationship is affected by many variables, the curve may be expected to be of the general form

$$f_c = A\,e^{BV}$$

where f_c = equivalent cube strength
e = base of natural logarithms
V = pulse velocity
and A and B are constants.

Hence a plot of log cube strength against pulse velocity is linear for a particular concrete (32).

3.3.2 *Practical factors influencing measured results*

There are many factors relating to measurements made on in-situ concrete which may further influence results.

3.3.2.1 Temperature. The operating temperature ranges to be expected in temperate climates are unlikely to have an important influence on pulse velocities, but if extreme temperatures are encountered their effect can be estimated from Figure 3.10. These factors are based on work by Jones and Facaoaru (33) and reflect possible internal microcracking at high temperatures and the effects of water freezing within the concrete at very low temperatures.

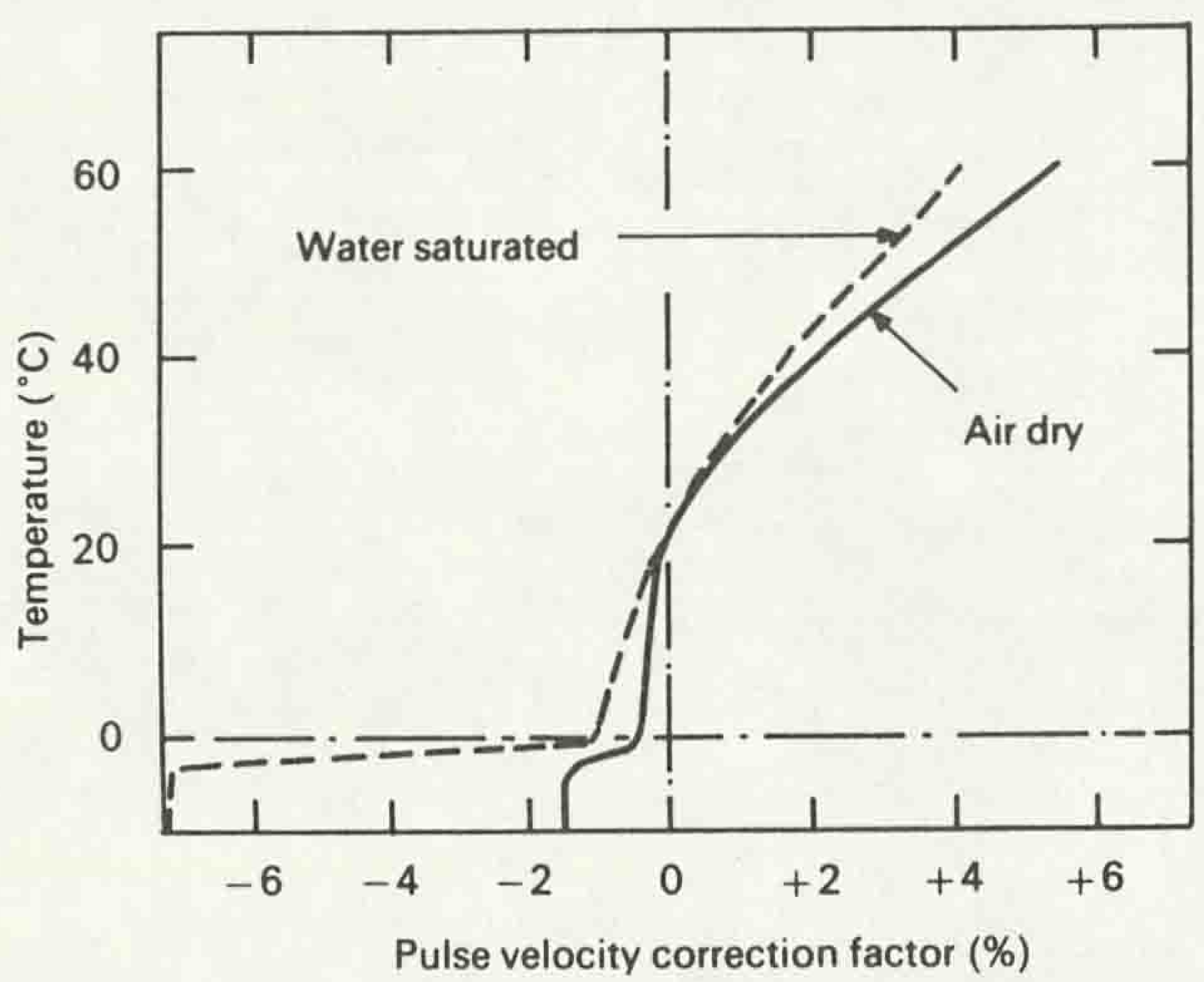


Figure 3.10 Effect of temperature (based on ref. 33).

3.3.2.2 Stress history. It has been generally accepted that the pulse velocity of laboratory cubes is not significantly affected until a stress of approximately 50% of the crushing strength is reached. This has been confirmed by Bungey (34) who has also shown from tests on beams that concrete subjected to flexural stress shows similar characteristics. At higher stress levels, an apparent reduction in pulse velocity is observed due to the formation of internal microcracks which will influence both path length and width.

It has been clearly shown that under service conditions in which stresses would not normally exceed $\frac{1}{3}$ cube strength the influence of compressive stress on pulse velocity is insignificant, and that pulse velocities for prestressed concrete members may be used with confidence. Only if a member has been seriously overstressed will pulse velocities be affected. Tensile stresses have been found to have a similarly insignificant effect, but potentially cracked regions should be treated with caution, even when measurements are parallel to cracks, since these may reduce path widths below acceptable limits.

3.3.2.3 Path length. Pulse velocities are not generally influenced by path length provided that this is not excessively small, in which case the heterogeneous nature of the concrete may become important. Physical limitations of the time-measuring equipment may also introduce errors where short path lengths are involved. These effects are shown in Figure 3.11, in which a laboratory specimen has been incrementally reduced in length by sawing. BS 4408 pt. 5 (26) recommends minimum path lengths of 100 mm and

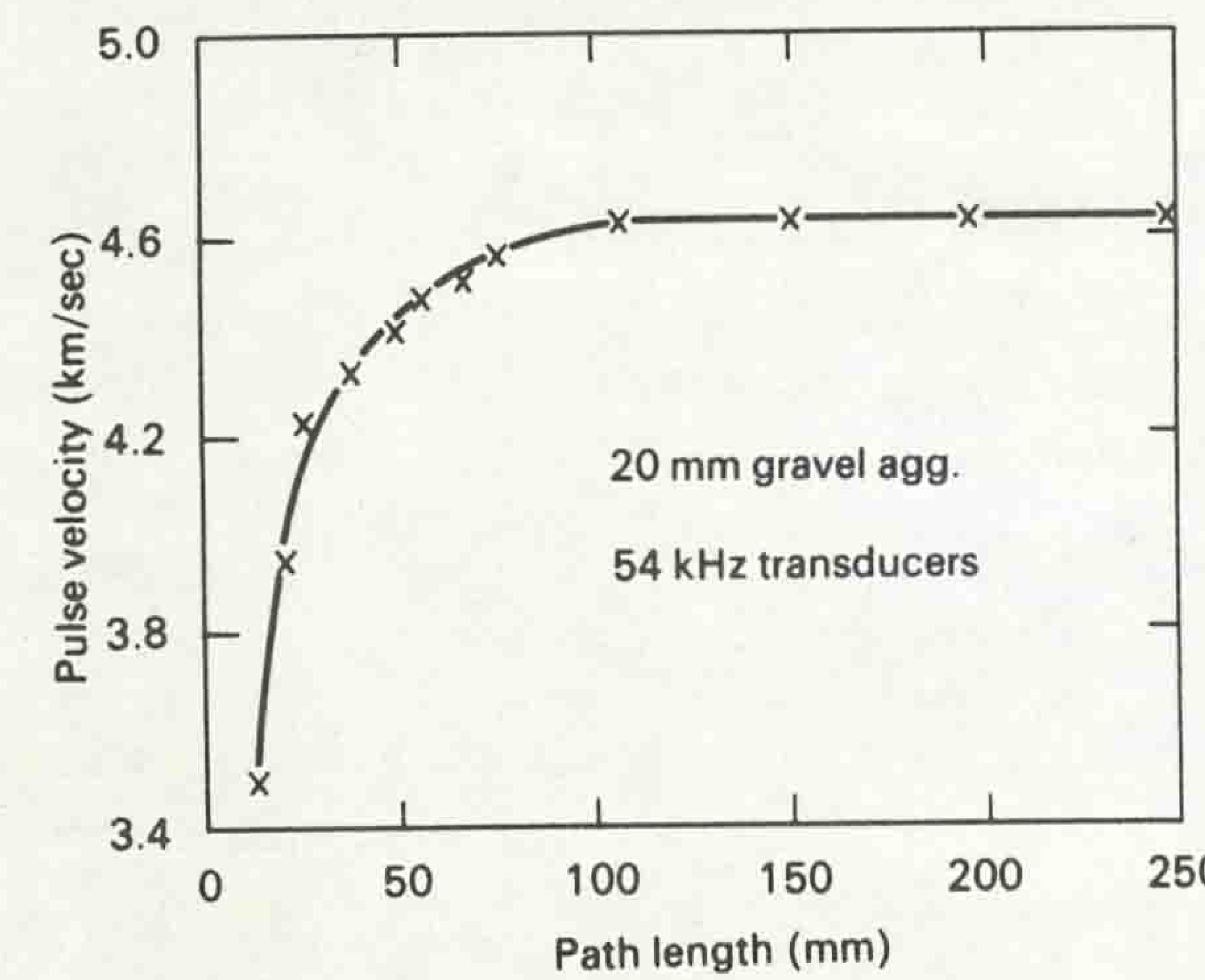


Figure 3.11 Effect of short path length (based on ref. 34).

150 mm for concrete with maximum aggregate sizes of 20 and 40 mm respectively. It also recommends that for unmoulded surfaces a minimum length of 150 mm should be adopted for direct, or 400 mm for indirect, readings.

There is evidence (17) that the measured velocity will decrease with increasing path length, and a typical reduction of 5% for a path length increase from approximately 3 m to 6 m is reported. This is because attenuation of the higher frequency pulse components results in a less clearly defined pulse onset. The characteristics of the measuring equipment are therefore an important factor. If there is any doubt about this, it is recommended that some verification tests are performed, although in most practical situations path length is unlikely to present a serious problem.

3.3.2.4 Moisture conditions. The pulse velocity through saturated concrete may be up to 5% higher than through the same concrete in a dry condition, although this influence will be less for high-strength than for low-strength concretes. The effect of moisture condition on both pulse velocity and concrete strength is thus a further factor contributing to calibration difficulties, since the moisture content of concrete will generally decrease with age. A moist specimen shows a higher pulse velocity, but lower measured strength than a comparable dry specimen, so that drying out results in a decrease in measured pulse velocity relative to strength. This effect is well illustrated by the results in Figure 3.12 which relate to otherwise identical

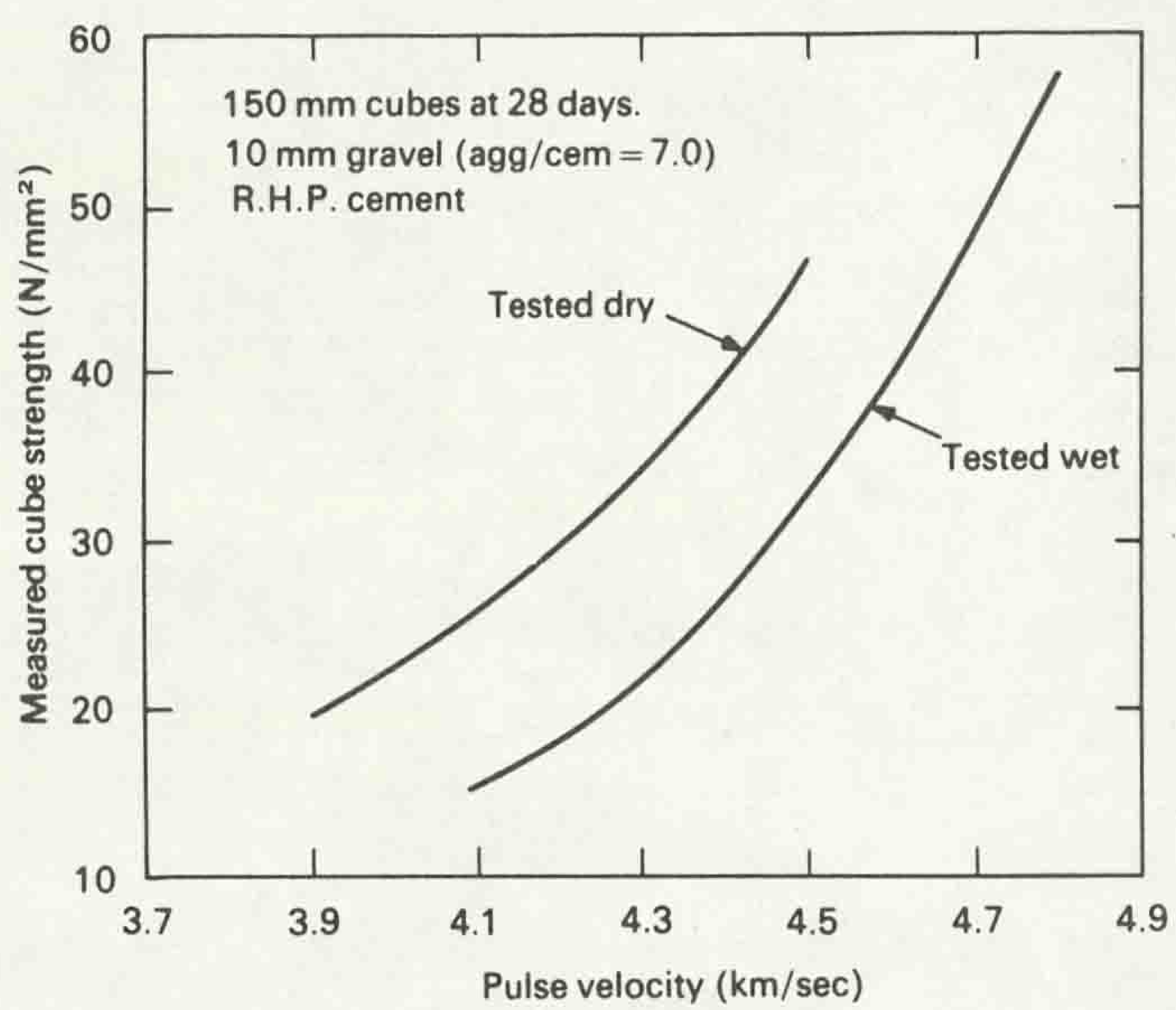


Figure 3.12 Effect of moisture conditions (based on ref. 34).

laboratory specimens, and demonstrates the need to correlate test cube moisture and structure moisture during strength calibration. It is thus apparent that strength correlation curves are of limited value for application to in-place concrete unless based on the appropriate maturity.

Tomsett (9) has presented an approach which permits calibration for "actual" in-situ concrete strength to be obtained from a correlation based on standard control specimens. The relationship between specimens cured under different conditions is given as

$$\log_e \frac{f_1}{f_2} = k f_1 (V_1 - V_2)$$

where f_1 is the strength of a "standard" saturated specimen
f_2 is the "actual" strength of the in-situ concrete
V_1 is the pulse velocity of the "standard" saturated specimen
V_2 is the pulse velocity of the in-situ concrete

and k is a constant reflecting compaction control (a value of 0.015 is suggested for normal structural concrete, or 0.025 if poorly compacted). This effect is illustrated by Figure 3.13, which is based on Tomsett's work (9). For any given curing conditions, it is possible to draw up a strength/pulse velocity relationship in this way, and similar members in a structure can be compared

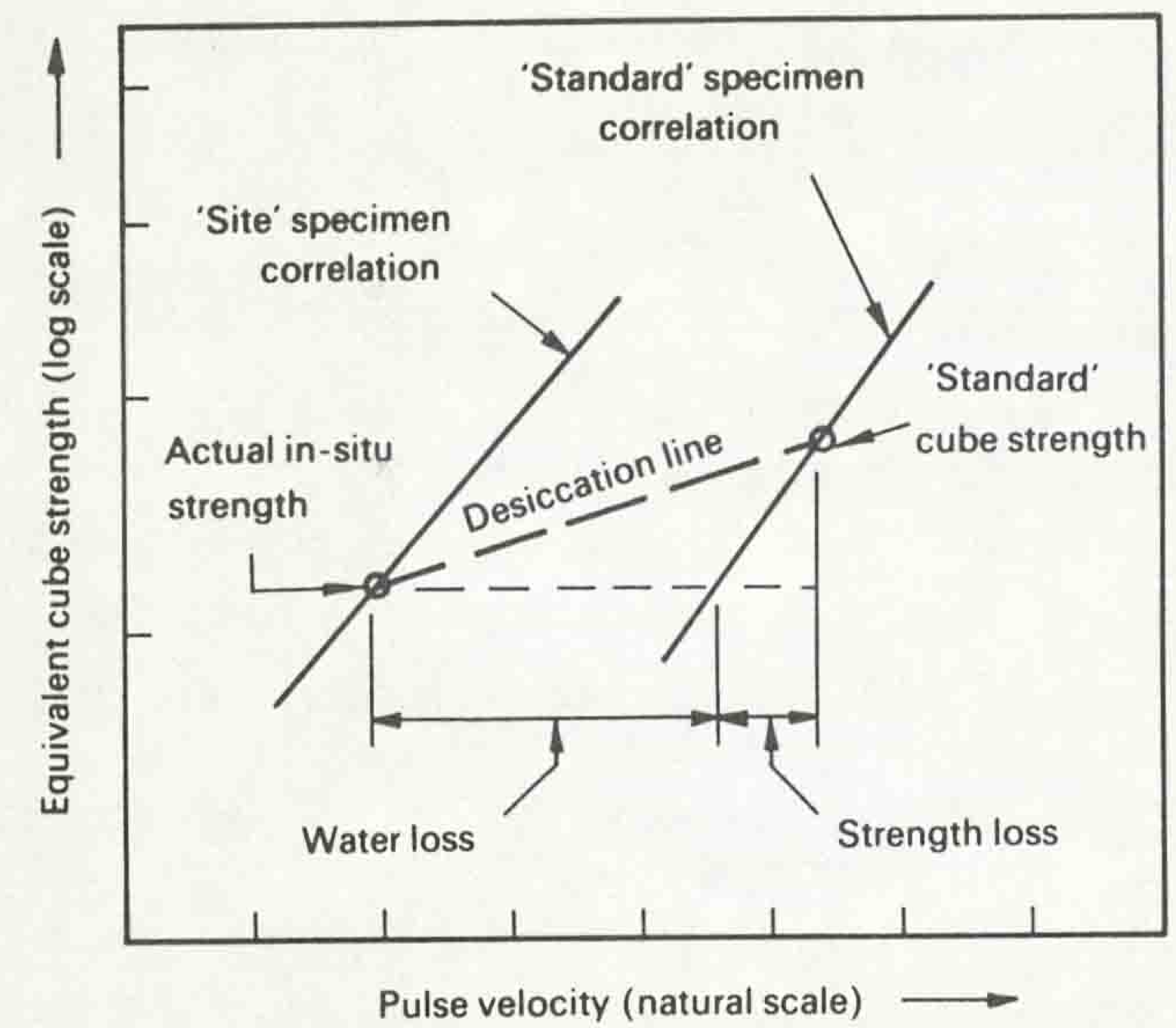


Figure 3.13 Desiccation line method (based on ref. 9).

from a single correlation which may be assumed to have the same slope as the "standard" saturated specimen relationship. This simple approach allows for both strength and moisture differences between in-situ concrete and control specimens. However, a direct strength assessment of a typical reference specimen of in-situ concrete is still required if the relationship is to be used for other than comparative applications.

3.3.2.5 Reinforcement. Reinforcement, if present, should be avoided if at all possible, since considerable uncertainty is introduced by the higher velocity of pulses in steel coupled with possible compaction shortcomings in heavily reinforced regions. The pulse velocity in an infinite steel medium is close to

5.9 km/sec, but this has been shown to decrease with bar diameter to as little as 5.1 km/sec along the length of a 10 mm reinforcing bar in air (34). The diameter of the reinforcing bar is thus a factor in making any adjustments for the presence of steel, and in the author's opinion, bars of 12 mm or less may safely be ignored irrespective of quantity or location relative to pulse path. There are two principal cases to be considered:

(*a*) *Axis of bars parallel to pulse path*
As shown in Figure 3.14, if a bar is sufficiently close to the path, the first wave to be received may have travelled along the bar for part of its journey.

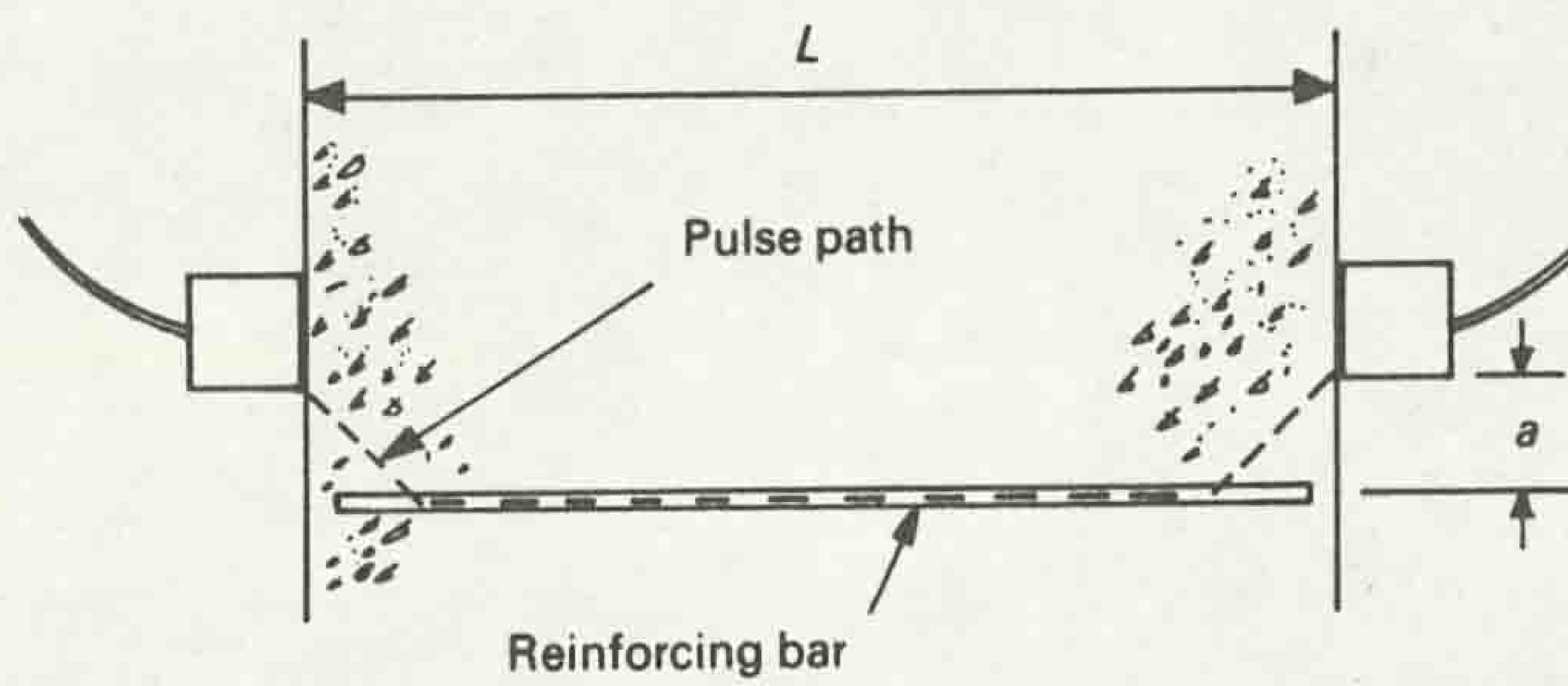


Figure 3.14 Reinforcement parallel to pulse path.

BS 4408 pt. 5 suggests that a relationship of

$$V_c = \frac{2aV_s}{\sqrt{4a^2 + (TV_s - L)^2}} \quad \text{when} \quad V_s \geqslant V_c$$

is appropriate, where V_s = pulse velocity in steel bar
and V_c = pulse velocity in concrete

and that this effect disappears when

$$\frac{a}{L} > \frac{1}{2}\sqrt{\frac{V_s - V_c}{V_s + V_c}}$$

hence steel effects may be significant when $a/L < 0.15$ in high quality concrete or < 0.25 in low quality material. If a value of V_s of 5.5 km/sec is adopted, as suggested in BS 4408, the typical correction factors are as shown in Figure 3.15.

Chung (35) has however shown that for pulses travelling in the direction of the axis of reinforcing bars through a steel/concrete medium, the effective velocity is considerably less than the theoretical value and is influenced by bar diameter. Chung's results give an effective measured pulse velocity of

$$V_e = 5.90 - 10.4\,(5.90 - V_c)/d$$

for 54 kHz pulses travelling along a bar embedded in concrete,

where V_c = true velocity in concrete
and d = bar diameter.

This relationship suggests that as bar diameter decreases, the BS 4408 recommendations increasingly overestimate the reinforcement influences, and this feature is also illustrated in Figure 3.15.

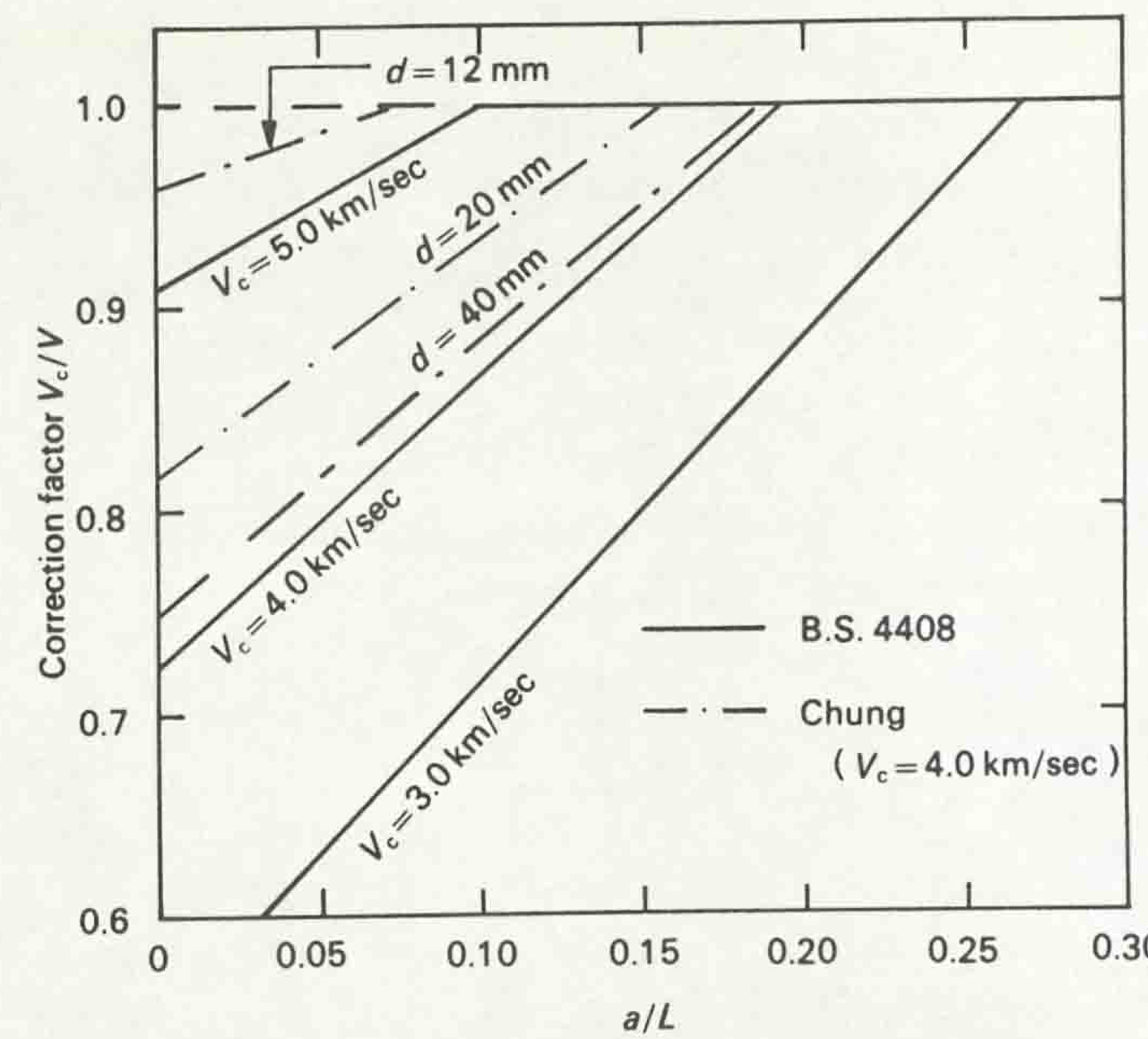
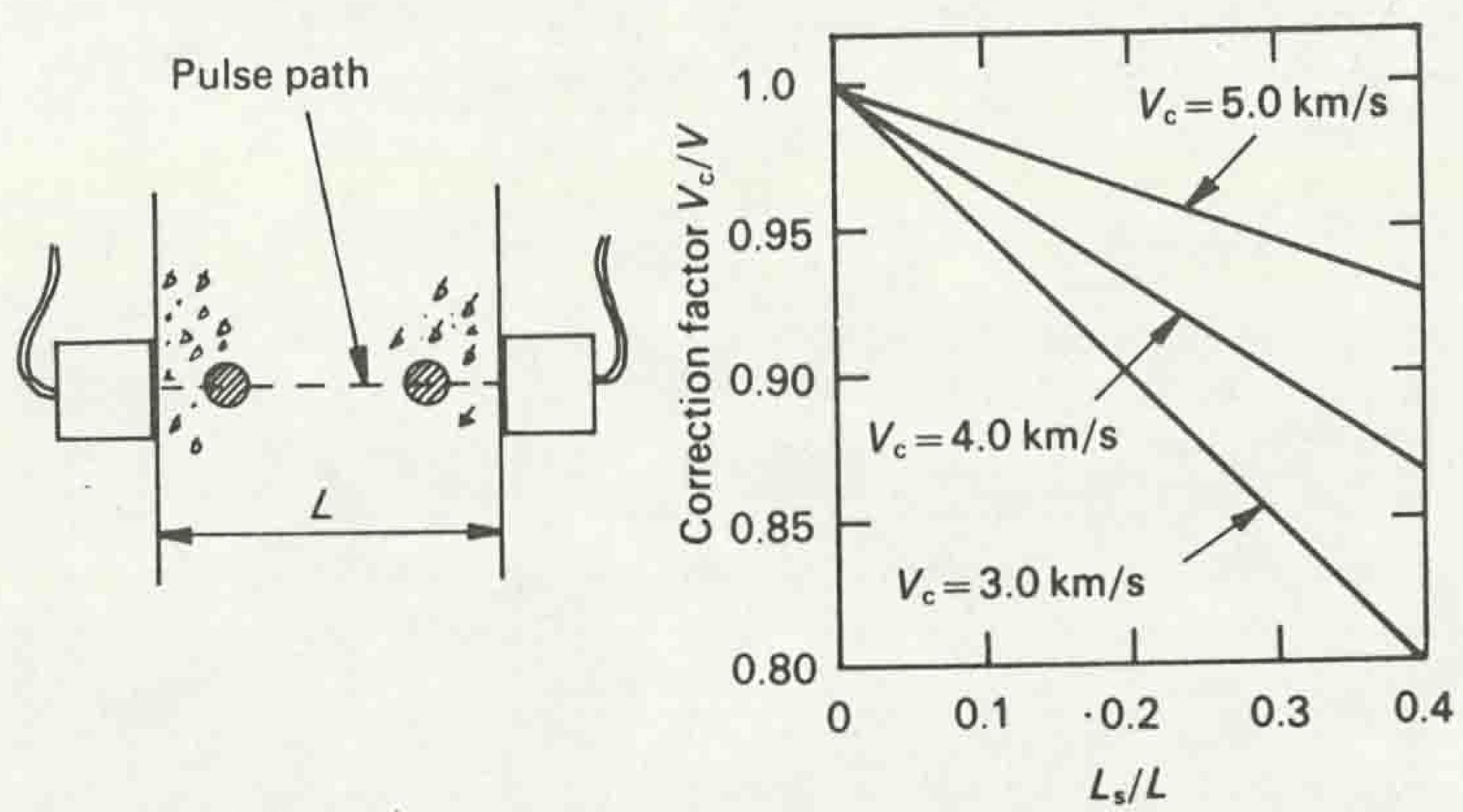


Figure 3.15 Reinforcement correction factors (based on refs. 26 and 35).

Such corrections must be treated with caution, especially since it is essentially the pulse through the concrete surrounding the bar which is being measured, rather than the body of the material, and the efficiency of bond between steel and concrete may affect the influence of the steel.

(*b*) *Axis of bars perpendicular to the pulse path*
For the situation shown in Figure 3.16(*a*), if the total path length through steel across the bar diameters is L_s, the maximum possible steel effect is given by Figure 3.16(*b*) for varying concrete qualities, where V_c is the true velocity in the concrete and V is the apparent measured value. Although there is an allowance for diameter there is little doubt that these correction factors are considerably overestimated, especially for small bars, because of the

significant effects of diameter on velocity. This is coupled with the fact that a very small proportion of the path length is through the full bar diameter, and it has been suggested (34) that a reduced "effective" diameter could be used to assess the necessary corrections.



(a) Path through transverse reinforcement

(b) Maximum correction factors

Figure 3.16 Reinforcement transverse to pulse path (based on ref. 26).

3.4 Applications

The applications of pulse velocity measurements are so wide-ranging that it would be impossible to list or describe them all. The principal applications are outlined below—the method can be used both in the laboratory and on site with equal success.

3.4.1 *Laboratory applications*

The principal laboratory applications lie in the monitoring of experiments which may be concerned either with material or structural behaviour. These include strength development or deterioration in specimens subjected to varying curing conditions, or to aggressive environments as described by Malhotra (17). The detection of the onset of micro-cracking may also be valuable during loading tests on structural members, although the method is relatively insensitive to very early cracking. For applications of this nature, the equipment is most effective if connected to a continuous recording device with the transducers clamped to the surface, thus removing the need for repeated application and associated operator errors.

3.4.2 *In-situ applications*

The wide-ranging and varied applications do not necessarily fall into distinct categories, but are grouped below according to practical aims and requirements.

3.4.2.1 Measurement of concrete uniformity. This is probably the most valuable and reliable application of the method in the field. There are many published reports of the use of ultrasonic pulse velocity surveys to examine the strength variations within members (6, 9, 34) as discussed in Chapter 1. The statistical analysis of results, coupled with the production of pulse velocity contours for a structural member, may often also yield valuable information concerning variability of both material and construction standards. Readings should be taken on a regular grid over the member. Typical pulse velocity contours for a beam constructed from a number of batches are shown in Figure 3.17.

Tomsett (9) has suggested that for a single site-made unit constructed from a single load of concrete, a pulse velocity coefficient of variation of 1.5% would represent good construction standards, rising to 2.5% where several loads or a number of small units are involved. A corresponding typical value of 6–9% is also suggested for similar concrete throughout a whole structure. An analysis of this type may therefore be used as a measure of construction quality, whilst the location of substandard areas can be obtained from the "contour" plot. The plotting of pulse velocity readings in histogram form may also prove valuable, since concrete of good quality will provide one clearly defined peak in the distribution (see section 1.5.2.1). Used in this way, ultrasonic pulse velocity testing could be regarded as a form of control testing, although the majority of practical cases in which this method has been used are related to suspected construction malpractice or deficiency of concrete supply. A survey of an existing structure will reveal and locate such features, which may not otherwise be detected. Although it is preferable to perform such surveys by means of direct readings across opposite faces of the member, Tomsett (36) has reported the successful use of indirect readings for comparison and determination of substandard areas of floor slabs.

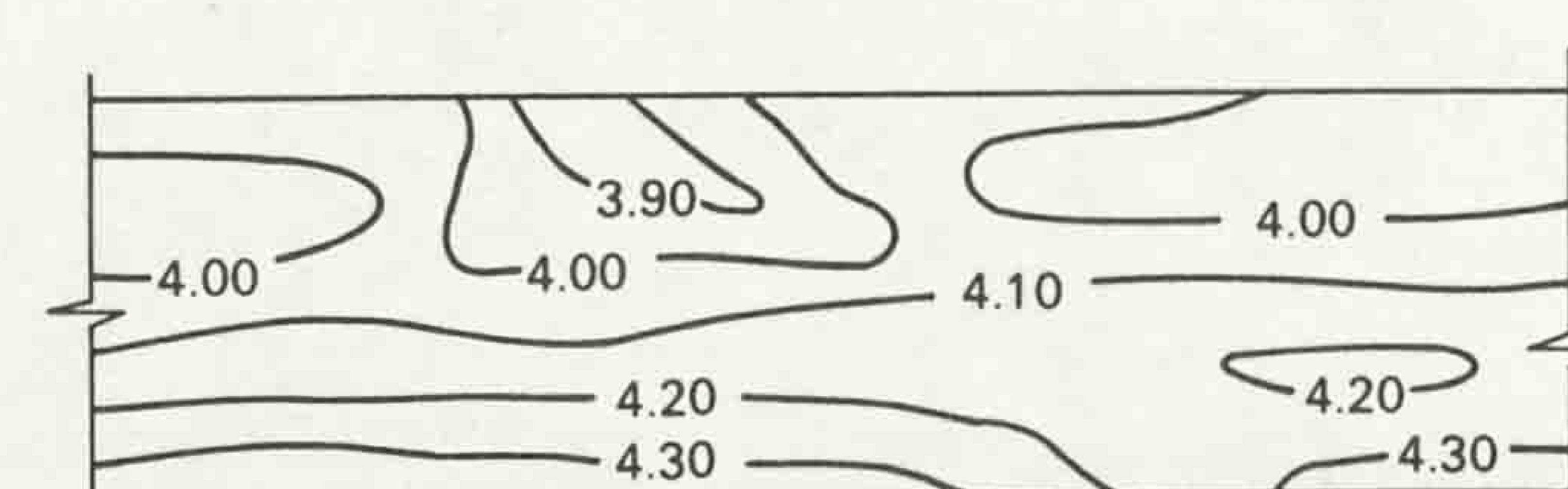


Figure 3.17 Typical pulse velocity beam contours.

Decisions concerning the seriousness of defects suggested by surveys of this type will normally require an estimate of concrete strength. As indicated in section 3.4.2.3, a reliable estimate of absolute strength is not normally possible, but if the mean strength of the supply is known, the relationship $f_c = kV^4$ has been found satisfactory for estimating relative values over small ranges (9). Failing this, it will be necessary to resort to a more positive semi-destructive method, such as core sampling, to obtain strength values, with the locations determined on the basis of the ultrasonic contour plot.

3.4.2.2 Detection of cracking and honeycombing. A valuable application of the ultrasonic pulse velocity technique which does not require detailed correlation of pulse velocity with any other property of the material is in the detection of honeycombing and cracking. Since the pulse cannot travel through air, the presence of a crack or void on the path will increase the path length (as it goes around the flaw) and a longer transit time will be recorded. The apparent pulse velocity thus obtained will be lower than for the sound material. Since the pulses will travel through water it follows that this philosophy will apply only to cracks or voids which are not water-filled. Tomsett (9) has examined this in detail and concluded that whilst water-filled cracks cannot be detected, water-filled voids will show a lower velocity than the surrounding concrete. Voids containing honeycombed concrete of low pulse velocity will behave similarly.*

In crack detection and measurement, even micro-cracking of concrete will be sufficient to disrupt the path taken by the pulses, and Bungey (34) has shown that at compressive stresses in excess of 50% of the cube crushing strength, the measured pulse velocity may be expected to drop due to disruption of both path length and width. If the velocity for the sound concrete is known it is therefore possible to detect overstressing, or the onset of cracking may be detected by continual monitoring during load increase.

An estimate of crack depths may be obtained by the use of indirect surface readings as shown in Figure 3.18. In this case, where the transducers are equidistant from a known crack, if the pulse velocity through sound concrete is V km/sec, then:

Path length without crack $= 2x$

Path length around crack $= 2\sqrt{x^2+h^2}$

Surface travel time without crack $= \dfrac{2x}{V} = T_s$

Travel time around crack $= \dfrac{2\sqrt{x^2+h^2}}{V} = T_c$

* See note, p. 59.

and it can be shown that

$$\text{crack depth, } h = x\sqrt{\left(\frac{T_c^2}{T_s^2}-1\right)}$$

An accuracy of $\pm 15\%$ can normally be expected, and this approach may be modified for applications to other situations as necessary.

Amon and Snell (37) have also described a number of case histories in which ultrasonic techniques have been used to monitor epoxy grout repairs to concrete based on the principle that poor bond or compaction will hinder the passage of pulses.

The location of honeycombing is best determined by the use of direct measurements through the suspect member, with readings taken on a regular grid. If the member is of constant thickness, a "contour map" of transit times will readily show the location and extent of areas of poor compaction.

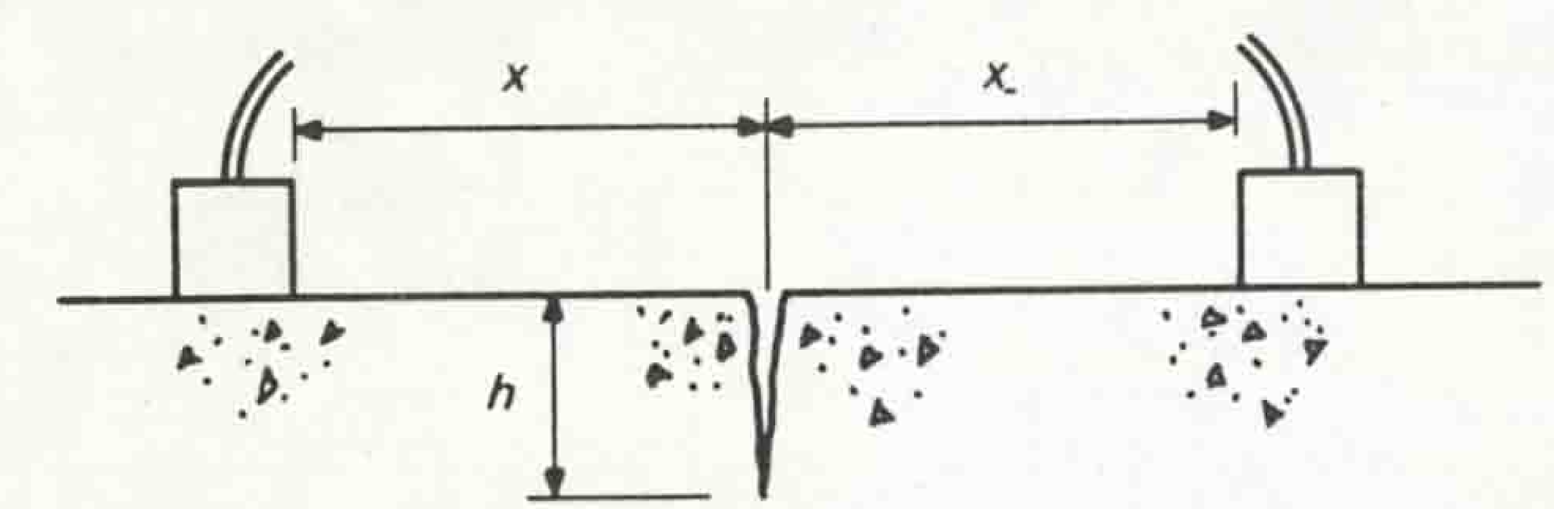


Figure 3.18 Crack depth measurement.

3.4.2.3 Strength estimation. Unless a suitable calibration curve can be obtained it is virtually impossible to predict the absolute strength of a body of in-situ concrete by pulse velocity measurements. Although it is possible to obtain reasonable correlations with both compressive and flexural strength in the laboratory, enabling the strength of comparable specimens to be estimated to $\pm 10\%$, the problems of relating these to in-situ concrete are considerable. If it is to be attempted, then the most reliable method is probably the use of cores to establish the calibration curve coupled with Tomsett's moisture correction (9). Bungey (34) has suggested that if a reliable calibration chart is available, together with good testing conditions, it may be possible to achieve 95% confidence limits on a strength prediction of $\pm 20\%$ relating to a localized area of interest. Expected within-member variations are likely to reduce the corresponding accuracy of overall strength prediction of a member to the order of $\pm 10\,\text{N/mm}^2$ at the $30\,\text{N/mm}^2$ mean level.

Although not perfect, there may be situations in which this approach may provide the only feasible method of in-situ strength estimation, and if this is necessary it is particularly important that especial attention is given to the relative moisture conditions of the calibration samples and the in-situ concrete. Failure to take account of this is most likely to cause an underestimate of in-place strength, and this underestimate may be substantial.

3.4.2.4 Assessment of concrete deterioration. Ultrasonics are commonly used in attempting to define the extent and magnitude of deterioration resulting from fire, mechanical or chemical attack. A general survey of the type described in section 3.4.2.1 will easily locate suspect areas, whilst a simple method for assessing the depth of fire or surface chemical attack has been suggested by Tomsett (9). In this approach it is assumed that the pulse velocity for the sound interior regions of the concrete can be obtained from unaffected areas, and that the damaged surface velocity is zero. A linear increase is assumed between the surface and interior to enable the depth to sound concrete to be calculated from a transit time measured across the damaged zone. For example if a time T is obtained for a path length L including one damaged surface zone of thickness t, and the pulse velocity for sound concrete is V_c it can be shown that the thickness is given by

$$t = (TV_c - L).$$

Although this provides only a very rough estimate of damage depth, it is reported that the method has been found to give reasonable results in a number of fire damage investigations.

Where deterioration of the member is more general, it is possible that pulse velocities may reflect relative strengths either within or between members. There is a danger that elastic modulus, and hence pulse velocity, may not be affected to the same degree as strength and caution should therefore be exercised when using pulse velocities in this way.

Whilst it may be possible to develop laboratory calibrations for a mix subjected to a specific form of attack or deterioration, as was attempted when evaluating high alumina cement decomposition in the United Kingdom (38), absolute strength predictions of in-situ deteriorated concrete must be regarded as unreliable. In-situ comparison of similar members to identify those which are suspect for subsequent load testing, has however been carried out successfully in the course of a number of H.A.C. investigations (39). This provides a relatively quick and cheap approach where a large number of precast units, for example, are involved. Long-term performance of concrete can also be monitored very successfully by conducting repetitive tests on the same element.

3.4.2.5 Measurement of layer thickness. This is essentially a development of the indirect reading method which is based on the fact that as the path length increases the pulse will naturally tend to travel through concrete at an

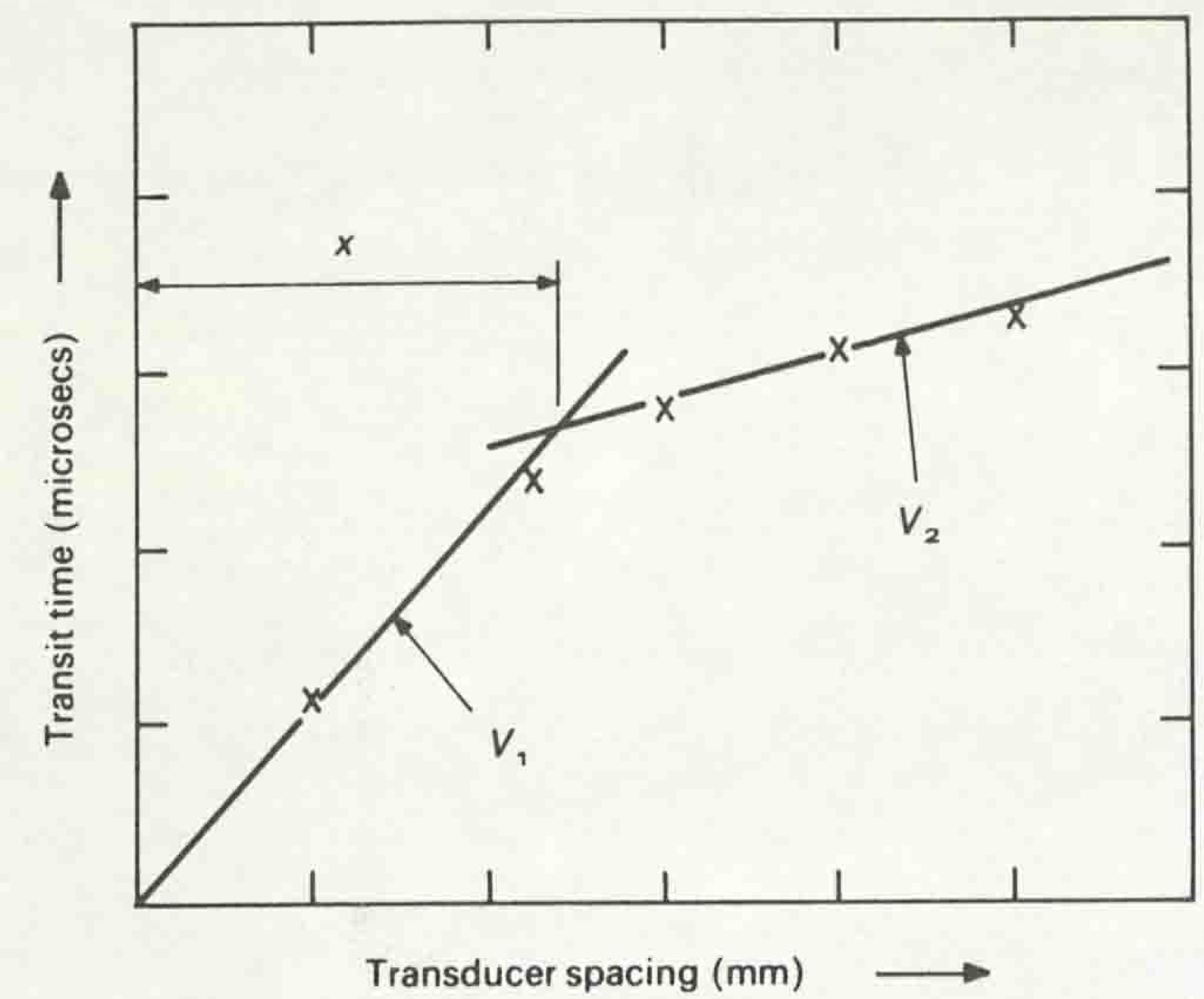


Figure 3.19 Layer thickness measurement.

increasing depth below the surface. This is particularly appropriate for application to slabs in which a surface layer of different quality exists due to construction, weathering, or other damage such as fire. The procedure is exactly as described for obtaining an indirect measurement (section 3.2.2.1). When the transducers are close together the pulse will travel in the surface layer only, whilst at greater spacings the path will include the lower layer. This effect will be shown by a discontinuity in the plot of transit time vs. transducer spacing, with the pulse velocities through the two layers having different slopes, as shown in Figure 3.19. The thickness t of the upper layer is related to the velocities V_1 and V_2, and the spacing x at which the discontinuity is observed, by the expression

$$t = \frac{x}{2}\sqrt{\frac{(V_2 - V_1)}{(V_2 + V_1)}}.$$

Whilst this is most suitable for a distinct layer of uniform thickness, the value obtained can be at best only an estimate, and it must be borne in mind that there will be a maximum thickness of layer that can be detected. Little information is available concerning the depth of penetration of indirect readings, and in view of the weakness of signal received using this method the results must be treated with care.

An approach has also been developed in the USA (40) to measure the thickness of pavement slabs, in which pulse velocity readings are combined with measurements of the resonant frequency of the slab. Using special equipment for this purpose which consists of a variable frequency transmitter, frequency counter, and dual beam oscilloscope it is claimed that an estimate of pavement thickness within $\pm 5\%$ can be obtained.

3.4.2.6 Measurement of elastic modulus. This is the property that can be measured with the greatest numerical accuracy. Values of pulse modulus can be calculated theoretically using an assumed value of Poisson's ratio to yield a value within $\pm 10\%$, or more commonly an estimate of dynamic modulus can be obtained from the reliable correlations with resonant frequency values. Whilst such measurements may be valuable in the laboratory when undertaking model testing, their usefulness on site is limited, although they may be used to provide an estimated static elastic modulus value for use in calculations relating to load tests.

3.4.2.7 Strength development monitoring. It has been well established that pulse velocity measurements will accurately monitor changes in the quality of the paste with time, and this may be usefully applied to the control of demoulding or stressing operations both in precasting works and on site. In this situation a specific pulse velocity/strength relationship for the mix, subject to the appropriate curing conditions, can be obtained and a safe acceptance level of pulse velocity established. In the same way, quality control of similar precast units may easily be undertaken.

3.5 Reliability and limitations

Ultrasonic pulse velocity measurement has been found to be a valuable and reliable method of examining the interior of a body of concrete in a truly non-destructive manner. Modern equipment is robust, reasonably cheap and easy to operate, and reliable even under site conditions, however it cannot be overemphasized that operators must be well trained and aware of the factors affecting the readings. It is similarly essential that results are properly evaluated and interpreted by experienced engineers who are familiar with the technique. For comparative purposes the method has few limitations, other than when two opposite faces of a member are not available. The method provides the only readily available method of determining the extent of cracking within concrete, however the use for detection of flaws within the concrete is not reliable when the concrete is wet.

Unfortunately the least reliable application is for strength estimation of concrete. The factors influencing calibrations are so many that even under ideal conditions with a specific calibration it is unlikely that 95% confidence limits of better than $\pm 20\%$ can be achieved for an absolute strength

prediction for in-place concrete. Whilst it is recognized that there may be some circumstances in which attempts must be made to use the method for direct strength prediction this is not recommended. It is far better that attention is concentrated upon the use of the method for comparison of supposedly similar concrete, possibly in conjunction with some other form of testing, rather than attempt applications which are recognized as unreliable and which will therefore be regarded with scepticism.

* The variation in pulse velocity due to experimental error is likely to be at least 2% notwithstanding variations in concrete properties, hence the size of a void must be sufficient to cause an increase in path length greater than 2% if it is to be detected. A given void is thus more difficult to detect as the path length increases, but the absolute minimum size of detectable defect will be set by the diameter of the transducer used.

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Paper 5

"Ultrasonic Pulse testing of high Alumina cement concrete on the site"

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On the site

by J. H. Bungey

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THE USE of ultrasonic pulse velocity measurements to assess concrete properties has been proposed for many years[1]. Early apparatus was complicated to use and restricted to the laboratory. However, in recent years portable equipment has been developed which is straightforward in operation and suitable for site use. Typical of such equipment is the 'PUNDIT' (Figure 1) which displays in digital form the time taken for an ultrasonic pulse to travel through the concrete between two transducers.

A major advantage of this testing approach is that it is possible to examine the interior of a concrete member, which is not possible with other non-destructive approaches such as surface hardness measurements. The velocity of the pulses through concrete can be shown to be related to the quality, although this velocity will normally lie in the range 3-5 km/sec for the complete range of practical concretes. Consequently great care must be taken both in measurement and correlation of results if a reliable estimate of material properties is to be obtained.

B.S. 4408 Pt. 5: 1974[2] details procedures to be followed for ultrasonic testing and highlights both the factors affecting the accuracy of measurement and those affecting correlation with concrete quality.

A programme of testing to examine these factors in more detail, with particular reference to practical difficulties of site measurement, has been in progress in the Department of Civil Engineering, University of Liverpool, for the past year. The results presented represent part of this investigation, which is continuing, and consider the application to high alumina cement concretes.

Testing procedure

'Direct' readings were obtained by applying 50mm diameter transducers to opposite faces of the concrete under test. To ensure that a strong pulse is transmitted through the concrete a couplant must be used, and for reasonably smooth concrete surfaces 'Vaseline' was found to be suitable and was used throughout the investigation. At each test point, the minimum value of pulse time obtained from at least two repeated readings was recorded and the path length measured carefully by calipers and steel rule. To enable effective interpretation of results, full information regarding test conditions, specimen details including reinforcement and moisture state, material composition and history were recorded.

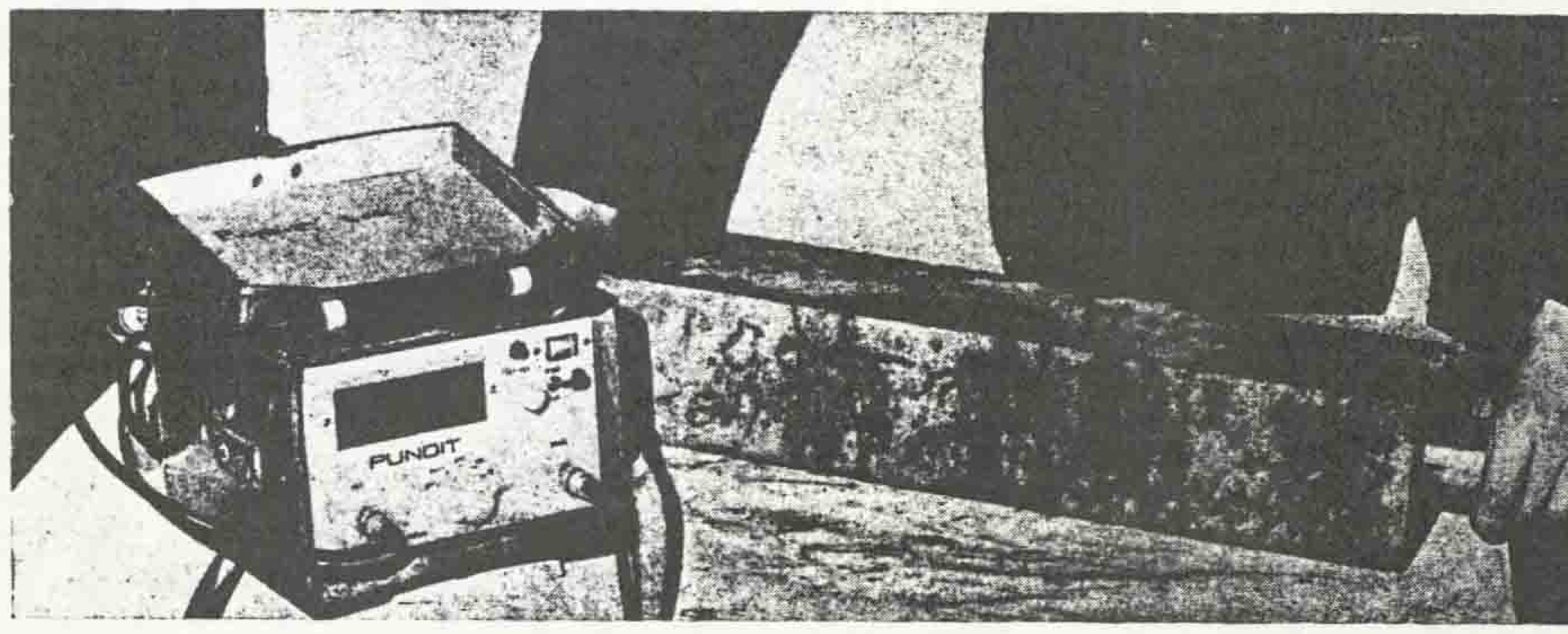

Figure 1. *The PUNDIT concrete tester in use.*

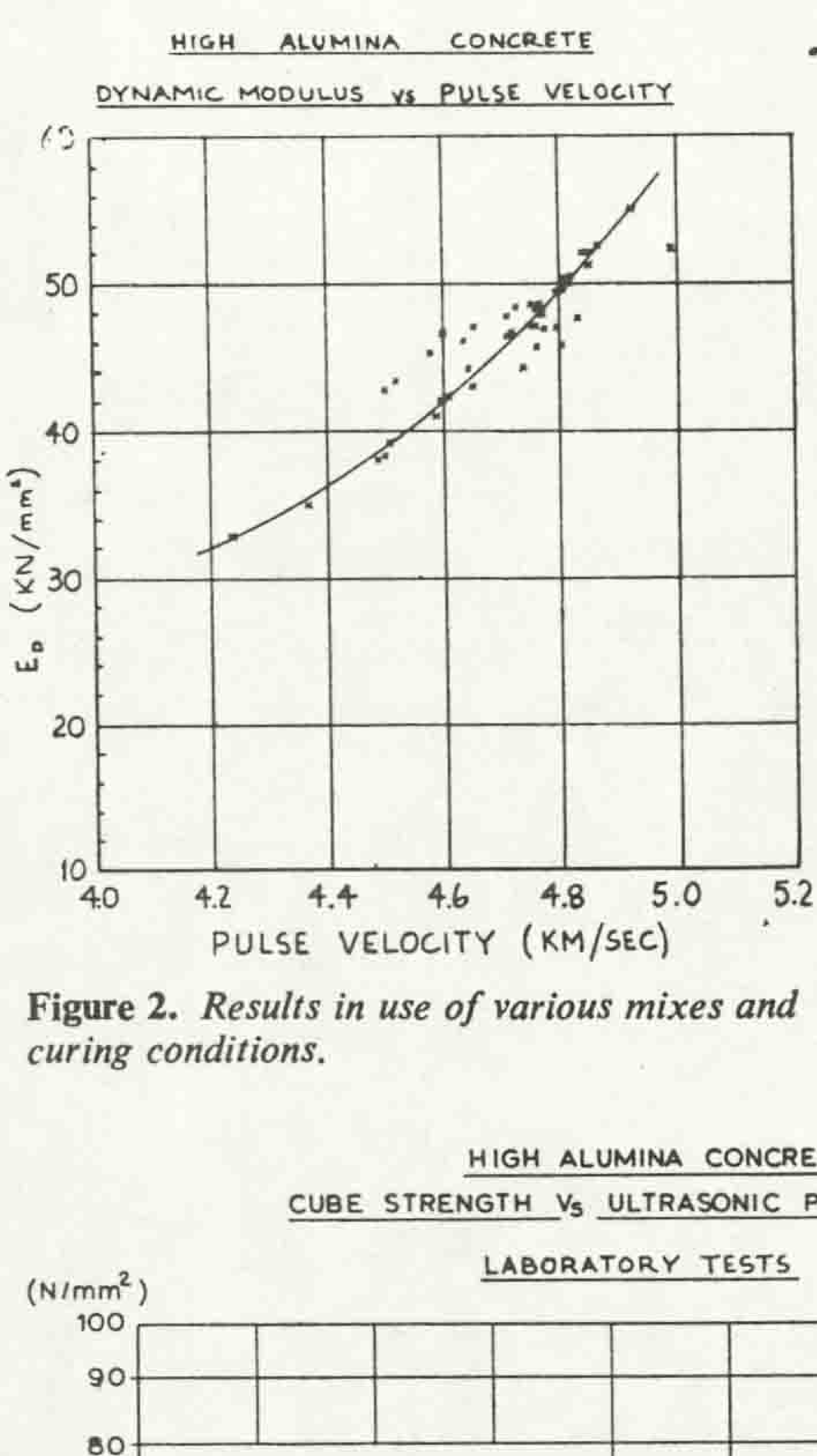

Figure 2. *Results in use of various mixes and curing conditions.*

Application to high alumina concrete

The pulse velocity in high alumina concrete is affected by factors which determine elastic stiffness. It has been found that the empirical relationship with dynamic modulus of elasticity suggested in B.S. 4408. Pt. 5: 1974[2] is closely applicable to high alumina concrete. Laboratory pulse velocity measurements of 500 $\times$100$\times$100mm prisms were compared with values of elastic modulus obtained by the 'resonance' test[5] on the same specimens, which represented a variety of mixes and curing conditions. The results shown in Figure 2 indicate a very small degree of scatter for a range of concrete strengths from 35-100 N/mm².

Although the relationship between

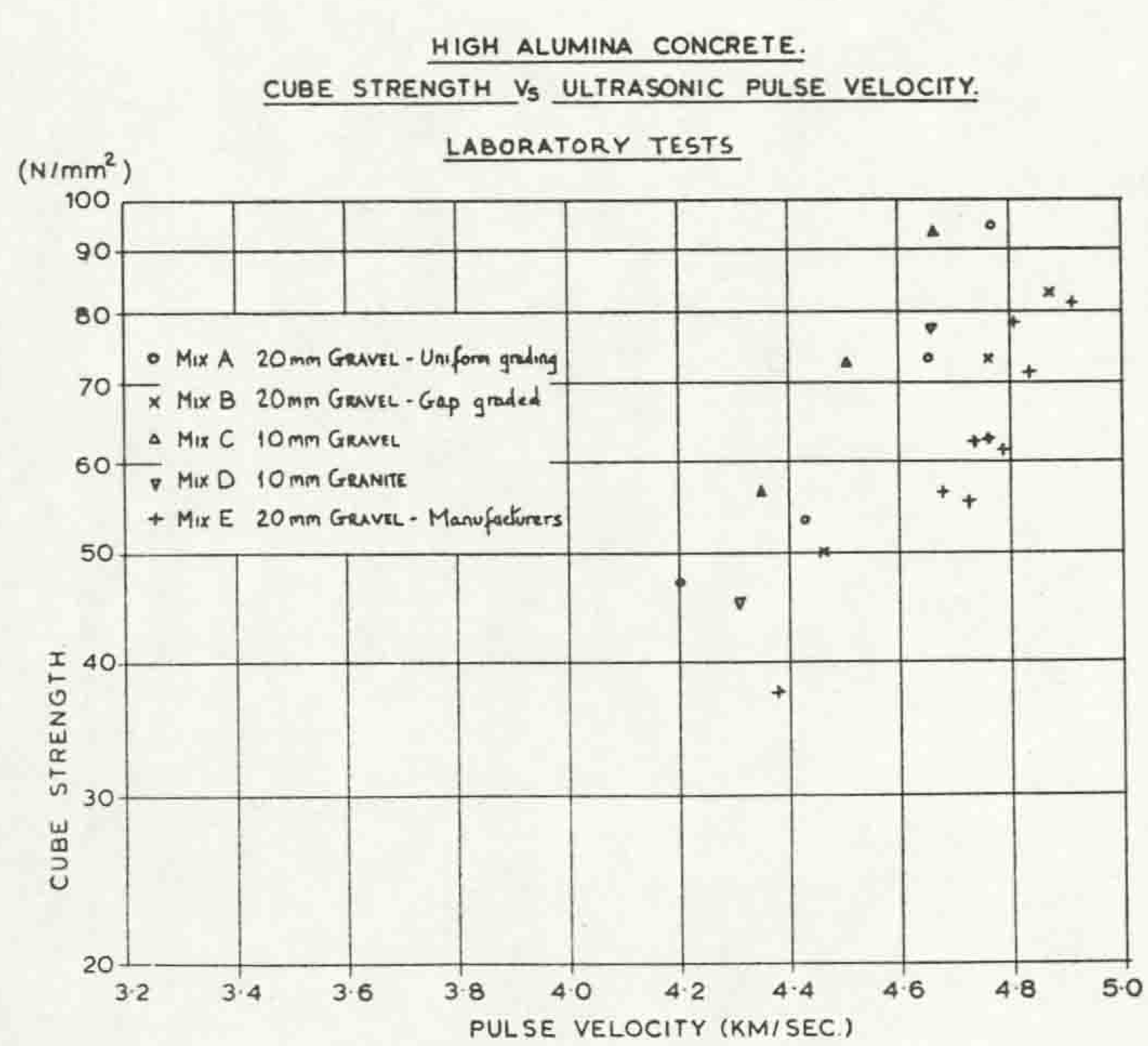

Figure 3. *Tests of five basic combinations of aggregate size, type and grading.*

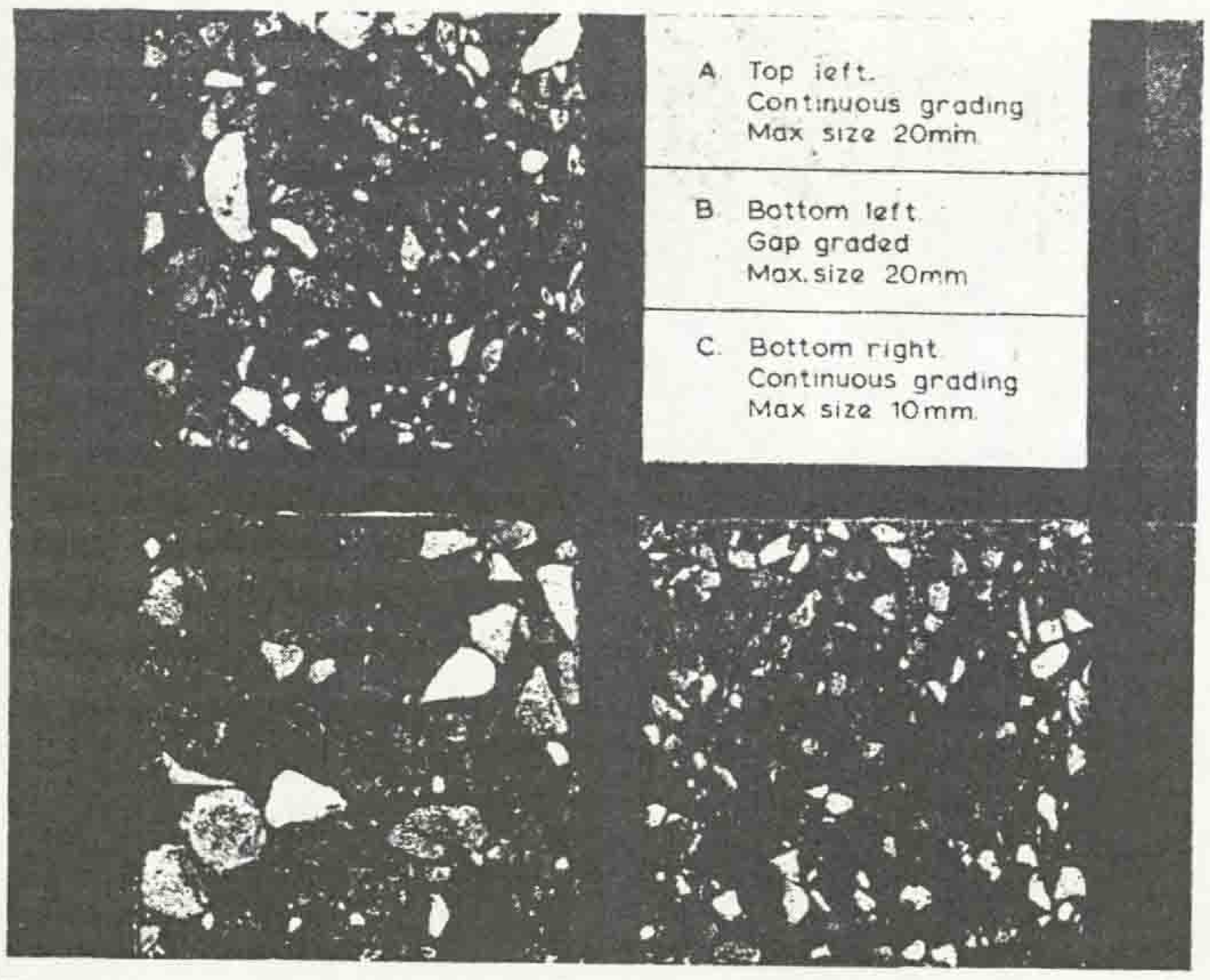

Figure 4. *The laboratory gravel mixes.*

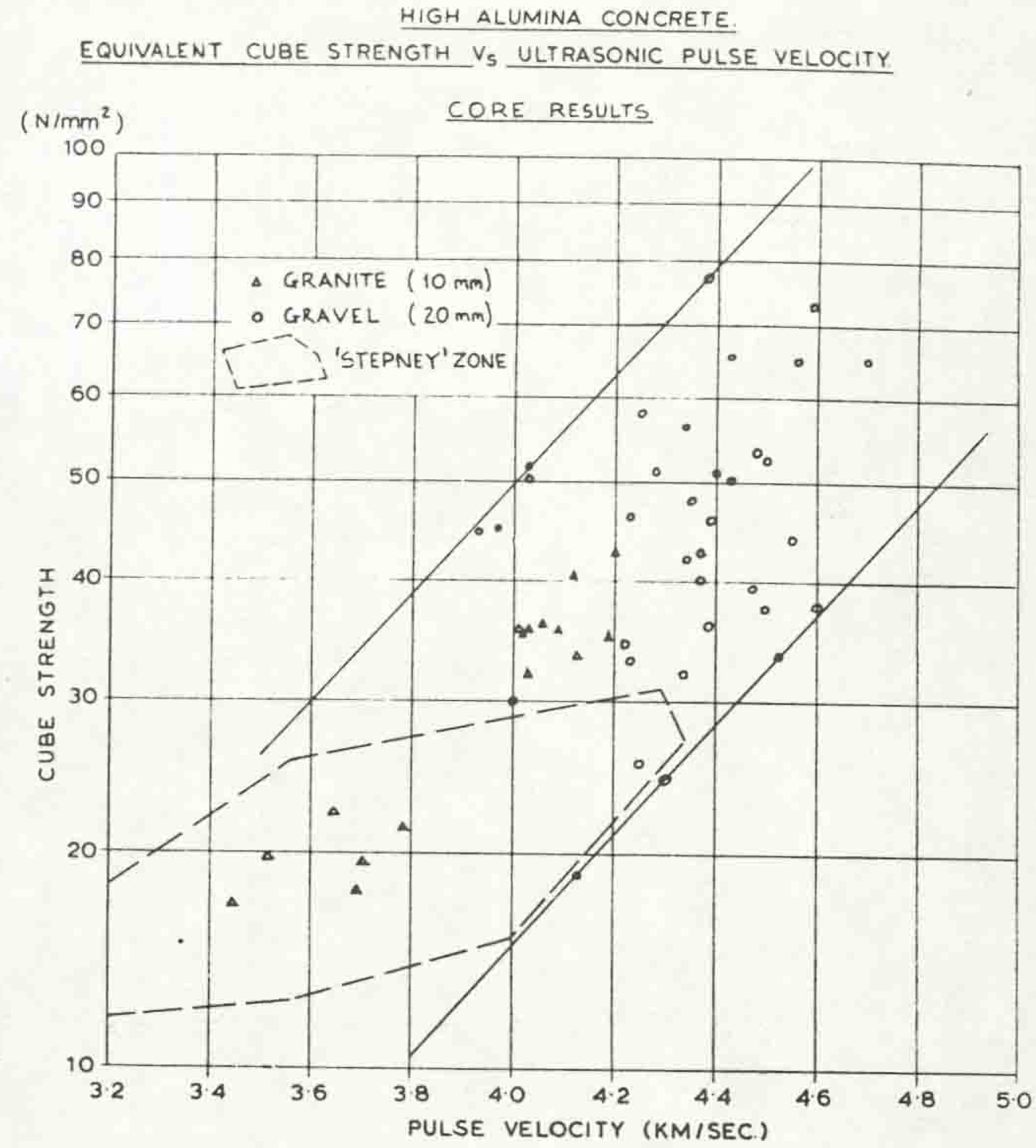

Figure 5. *Results from core tests of partially converted concrete.*

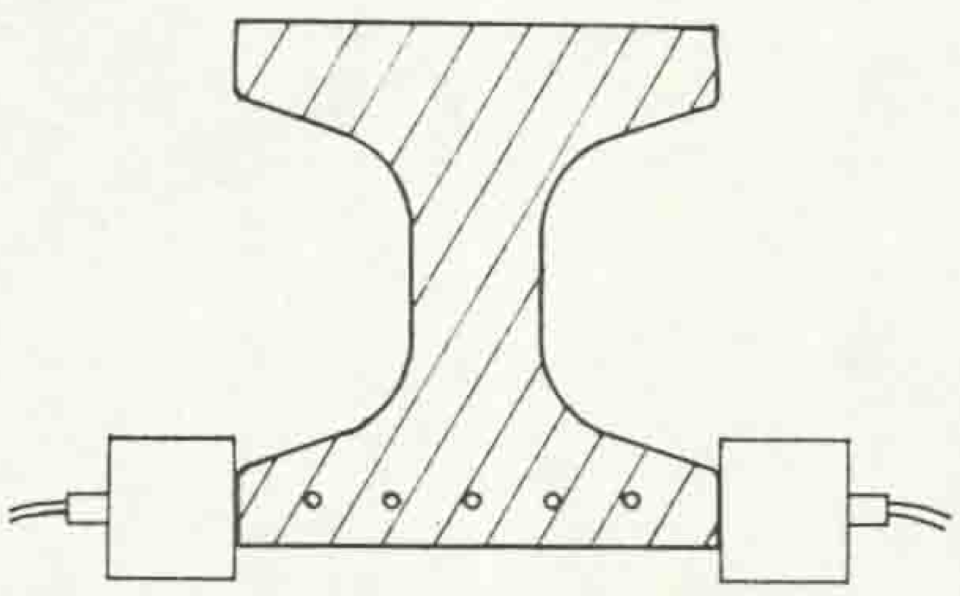

Figure 6. *Direct readings on small section beam.*

pulse velocity and modulus of elasticity is largely independent of concrete details, the correlation of pulse velocity with strength is not so simple, being related to aggregate type, size and grading, as well as mix design, age and curing conditions. It is generally accepted that these factors make the estimation of strength an operation to be treated with extreme caution. For Portland cement concretes the correlation curve is of the general form:

$$\text{Equivalent Cube Strength} = Ae^{BV}$$

where e = base of natural logarithms
V = pulse velocity
A and B are constants

hence a plot of log cube strength against velocity is linear for a particular concrete[3]. A limited series of laboratory tests on high alumina mixes indicate that such a relationship can also be applied to this material (Figure 3). A total of five basic combinations of aggregate size, type and grading were used, including a typical 'manufacturers' mix, with ultrasonic readings on $500 \times 100 \times 100$ prisms and crushing strength obtained from corresponding 100mm cubes. The required variations of strength were produced principally by water/cement ratio adjustments within practical limits, with overall proportions for the laboratory mixes being maintained at 1:1½:3. The three laboratory gravel mixes are illustrated in Figure 4 and the effects of grading are clearly discernible, with the gap-graded 20mm gravel (Mix B) corresponding the most closely to the typical manufacturers concrete (Mix E). Some evidence exists[1] that crushed rock aggregates will yield a higher pulse velocity than gravel for a particular concrete strength, and the results for 10mm gravel and granite (Mixes C and D) appear to confirm this.

It is noticeable also that the mixes containing the higher proportion of large aggregate yield the higher pulse velocities at a given strength level. These tests were all performed at ages of less than seven days on air-cured specimens, and it is anticipated that the results would yield different correlations if strength variations were produced by varying other factors, as is found for Portland cement concretes. The overall trend of this small number of results, however, indicates a general relationship of the form suggested, whilst the problems of strength estimation of an unknown concrete are demonstrated.

Effects of conversion

It is well known that the strength of high alumina concrete may be reduced by conversion.[4] In view of the increase in porosity accompanying this process it seems likely that this change would be reflected in pulse velocities. An indication that this may be possible was given by Jones and Gatfield[1] in 1955, but the practical problem of obtaining correlation charts for "converted" concrete has resulted in little quantitative information being available.

Attempts have been made to simulate the long term conversion effects in the laboratory by a variety of techniques which generally involve subjecting the green concrete to hot water curing. Whilst it is possible to produce dramatic strength reductions within a short time span, it has been found that this is not necessarily accompanied by a corresponding drop in pulse velocity. This has been indicated by a limited number of tests by the author where both static and dynamic modulii of elasticity were found to be virtually unchanged despite a 50% strength reduction, and this phenomenon has also been found in a more extensive investigation at Nottingham University by Dr B. Mayfield. It would appear therefore that very rapid conversion does not produce the same effects on the material as those occurring due to slower conversion rates.

Recourse must therefore be made to core and ultimate load tests if correlation is to be attempted between pulse velocity and 'converted' concrete in structures. Results have recently been obtained from core tests from a number of structures, the procedure adopted being to take insitu pulse velocity measurements as closely as possible to positions where cores were to be cut, which were supplemented by further velocity measurements on the cores immediately before crushing. In most cases comparable values were obtained despite the effects of capping material and increased moisture content, since cores were stored under water in accordance with B.S. 1881[5]. In the few cases where appreciable variations were found, results have been plotted on the basis of the higher pulse velocity, to yield the most conservative strength correlation. The maximum diameter of core was generally governed by the nature of the member, and results relate to 43mm cores in most instances, with a few at 34mm.

Information has been obtained for two basic types of coarse aggregate, 10mm granite chips and 20mm gravel, which are those most commonly to be found in practice. The results of these tests are shown in Figure 5 and are divided into these two categories.

The granite results relate to concrete from one source but are extracted from a variety of member types which have been subjected to a considerable range of environments. There can be little doubt that a good degree of correlation exists for this particular concrete despite the inaccuracies resulting from small size core testing.

The results for gravel are considerably more variable although this is not unexpected since a number of sources are involved, and in addition the larger aggregate size is likely to cause a greater scatter of core results. Few gravel concrete cores have yet been obtained by the author for very low cube strengths; it is interesting however to note the zone of results obtained by Dr Bate at the Building Research Establishment for such concrete during his investigation of the Stepney failure[6].

The upper and lower limits for all the results so far available have been drawn on Figure 5 and indicate that although a precise correlation is not possible to cover any mix, ultrasonic pulse velocity will offer a useful guide to the general strength range of a high alumina concrete which has undergone conversion.

The results for sound concretes given in Figure 3 can furthermore be shown to lie within these limits.

Factors affecting accuracy of field results

These may be divided into the two categories of accuracy of reading, and accuracy of correlation. It is important to consider the practical factors affecting the assessment of concrete by insitu pulse velocities related to equivalent cube strengths from core results.

Accuracy of readings

The insitu testing of small section beams produces particular problems in addition to those normally considered. Very often the choice lies between a very short path length across the web, which is difficult to measure accurately, or a longer path across flanges which may contain steel, and fall below the minimum recommended dimension transverse to the direction of pulse, which is about 80mm for a 50kHz pulse. It has been found that the longer path length across the flange yields more reliable information after correction for steel, with very short path lengths yielding appreciably lower pulse velocities. It has also been found that provided good coupling has been obtained, readings are not greatly influenced if the face of the transducer overhangs the concrete surface.

Readings must frequently be obtained under difficult conditions at considerable heights above floor level, and under such circumstances special care must be taken to ensure good contact between transducers and a smooth, plane concrete face. Any finishes applied to the member must be completely removed and path lengths must be measured very carefully, this being a particular problem in very deep members or those with a large bottom flange where calipers cannot be used to measure the top flange width.

Despite these problems, if care is taken and the recommendations of B.S.4408[2] are followed, it should be possible to estimate a velocity to $\pm 2\%$ from direct readings over a length of 200mm, although as the path length reduces the accuracy with which this can be measured may increase the range.

Accuracy of correlation

The correlation between pulse velocity in 500mm prism and the crushing strength of 100mm cubes for a particular mix, as shown in Figure 3, is markedly better than that obtained from insitu measurements related to core tests. Although the reduced accuracy of pulse velocity measurements will account partially for this, a major factor is likely to be the small size of cores, particularly where 20mm gravel mixes are concerned.

Pulse velocities measured through cores agreed surprisingly closely to those measured in the member despite the increased moisture content due to storage under water, and the effects of the high alumina mortar capping. It is generally accepted that increased moisture content increases the pulse velocity, but a few tests by the BRE[6] on high alumina cores to examine this proved inconclusive. To assess the possible effects of high alumina mortar capping on pulse velocities through cores, 100mm cubes of the mortar mix were subjected to the same curing conditions and tested at the same age as the capping. These indicated consistent pulse velocities of 3·9km/sec. Thus, velocities in low strength concrete may be increased, whilst for high strength concrete velocities are reduced by the capping. On the core sizes used, the capping material has generally represented about 12% of the path length through the core, thus in the extreme cases the overall core pulse velocity would be affected by no more than 2% due to this cause.

The accuracy of strength obtained from very small cores must always be treated with caution, and decisions based on these results should involve this consideration.

Conclusions

Results indicate that a degree of correlation between pulse velocity and strength does exist for high alumina concrete. Whilst curves relating to sound concrete can be easily obtained on laboratory specimens with a good degree of reliability, relationships for naturally converted concrete can only be estimated with the aid of core tests. It has been shown that for similar mixes subjected to varying environments it may be possible to obtain reasonable accuracy of correlation, but the wide range of materials and mixes used over the past 20 years makes scatter inevitable when multipurpose charts are plotted.

The value of this testing approach for the assessment of in situ high alumina concrete members lies in estimating a likely strength range for confirmation by more tedious destructive methods. Careful use of the technique can offer considerable advantages both in locating suspect areas for further attention and also for comparison of similar members, thus enabling the most effective use of core or load tests. It must be emphasised that the results presented are in no way exhaustive and the possibility of values outside the limits indicated must always be considered. Similarly, tests on a member relate only to the particular spots tested and there is always a danger of localised weak areas which may be missed even in a thorough investigation. In view of the problems encountered in efforts to accelerate the conversion process in the laboratory, any attempts to assess 'young' members in this way must be considered with great caution.

Because of the narrow range of velocities involved, great care must be exercised when testing under field conditions, with an awareness and understanding of all the factors liable to affect readings. It is therefore necessary that only a skilled and experienced operator is used if meaningful results are to be obtained. The accuracy of measured pulse velocity must be assessed for each situation, and interpretation of results must therefore be based on a detailed knowledge of the member, materials and testing conditions.

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Paper 6

"The Performance and Assessment of roofs and floors incorporating precast prestressed concrete"

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THE PERFORMANCE AND ASSESSMENT OF ROOFS AND FLOORS INCORPORATING PRECAST PRESTRESSED CONCRETE

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INTRODUCTION

A situation may arise from time to time, when it becomes necessary to estimate the structural capacity of members which are incorporated in buildings. Although the need for such assessments has long existed, the High Alumina Concrete problem (1) has, since 1974, provoked much interest in available testing methods. Many problems associated with the examination of concrete members under service conditions have been highlighted, especially where small section precast pretensioned concrete elements are used.

Although a large proportion of precast prestressed concrete members used in floors and roofs have been made with High Alumina Concrete in the past 25 years, a great many of the problems of testing are independent of the cement type. Furthermore, in the course of checking the condition of High Alumina Concrete members after conversion, a number of important shortcomings in design, construction and maintenance emerge. The examples reported in this paper have been encountered during the inspection of structures in the Merseyside area, but it is likely that they typify conditions throughout the United Kingdom.

TESTING PROCEDURES AVAILABLE

Whatever the cause of the investigation, the fundamental approach given in Figure 1 will apply, with the programme proceeding until the Engineer has sufficient information to reach a decision regarding his particular problem.

Visual Inspection

The first stage of any investigation must be visual inspection. This will immediately indicate if a member shows signs of distress under its normal loading. Excessive deflections, cracking or spalling may be sometimes evident, but equally important is to note the structural action of the member and likely degree of 'undesigned' action from finishes. Colour is a valuable guide in the assessment of fire damage, and localised chemical attack may exhibit itself as staining whilst more general attack may take the form of surface crumbling or spalling near reinforcement.

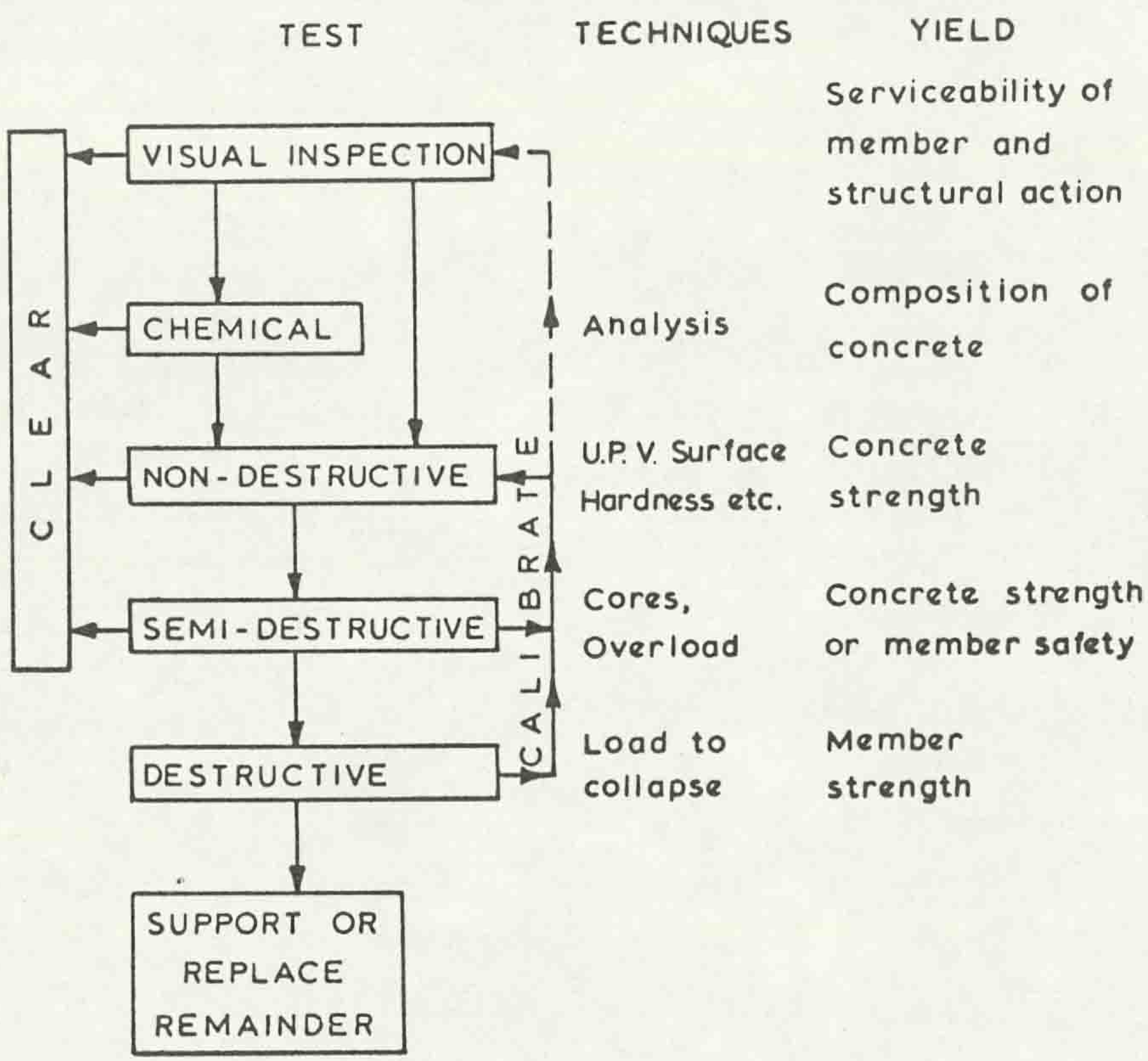

Figure 1 - Testing Procedures

Chemical Analysis

Whilst offering information on cement type and content, together with aggregate type and grading, chemical analysis cannot as yet offer information regarding concrete strength. It is therefore only when the concrete is seriously substandard from batching, or showing signs of excessive attack from chemicals that deductions can be made of low strength or durability.

Surface Hardness Measurements

The most common technique uses a rebound hammer, and is described in BS.4408 (2). Unfortunately no indication is given of the interior of the concrete member, and for concrete which is more than a few months old surface carbonation effects dominate. Calibration curves relating rebound number with strength must always be produced for the particular mix used. These must also incorporate factors such as the curing methods, stress state and slenderness of the member, hence this method has little practical use when considering precast prestressed units, other than comparison of 'identical' units.

Ultrasonic Pulse Velocity Measurements

Measurement of pulse velocities is discussed in BS.4408 (2). The pulse velocity through concrete is essentially related to the dynamic elastic modulus. Difficulties arise in obtaining reliable readings, and in estimating the effects of reinforcement, which make great care during testing essential. Correlation with strength is not simple, depending not only on mix properties, but also stress state. Many of these problems are discussed elsewhere (3) and are examined in more detail later.

Core Testing

Described in BS.1881 (4) core testing enables the concrete interior to be examined visually, and gives an estimate of compressive strength. Correction factors to allow for length/diameter ratio, and to convert to an equivalent cube strength are not precise, and for diameters greater than 75mm. it is seldom possible to expect accuracies better than ±20%.

Load Testing

Insitu load testing will demonstrate the ability of a member or structure to carry a small overload safely at a given time. Deflections can be compared with theory, and recovery checked, but the effects of non-structural finishes generally mean that little information is obtained regarding the reserve of strength. Where composite action is intended between precast members and an insitu slab, the strength prediction of the overall system is further hindered. Although the prestressed member and slab can both be assessed for strength, the efficiency of the composite action must be assumed. If there is any doubt regarding this, proof can only be obtained by an insitu test of a suitably isolated section of floor to a considerable overload.

Ultimate load testing is almost invariably restricted to individual members and, if the maximum information is to be gained, should preferably be carried out in the laboratory. However impracticable, this is the only reliable method of obtaining member strength, and is valuable in calibrating other tests.

NON-DESTRUCTIVE AND SEMI-DESTRUCTIVE STRENGTH ESTIMATION OF PRESTRESSED CONCRETE

In most cases it is the member strength that must be assessed. Testing to destruction is the only positive proof available concerning member strength, but this is obviously not feasible on anything but a very limited scale. Thus if an assessment of ultimate strength capacity is required of a large number of members, use must be made of the non-destructive or semi-destructive tests.

Considering first the principal non-destructive tests, both can be used for comparing apparently identical members, but surface hardness tests must be treated with great caution in view of the dominating effect of skin hardening. Ultrasonics offer a more reliable means of comparison which involves the interior of the member, although this introduces the added complication of steel effects. With either test, the major problem is calibration, when dealing with a concrete of unknown composition and characteristics. Since surface hardness testing is of doubtful value for the types of member considered in this paper, it is more fruitful to consider Ultrasonics in more detail.

Ultrasonic Testing of Prestressed Members

In addition to the general problems of ultrasonic testing, when dealing with prestressed members the problem of microcracking becomes important. If pulse velocities are measured transversely across the faces of a cube subjected to compression, a small reduction in pulse velocity occurs at stresses in excess of about 35% of the crushing strength. Tests on cubes with 20mm. aggregate, show that this effect can become quite significant at stresses above 60% of the crushing strength, as in Figure 2.

Since the stress distribution in a cube is complex, attempts were made to perform similar tests on prisms subjected to longitudinal compression. A number of prisms failed suddenly at stresses well below 50% of the corresponding cube strength without significant change in pulse velocities, however some tests were more successful, and results shown in Figure 2 were obtained. It can be seen here that pulse velocities are reduced significantly at stresses above 40% of the cube strength.

The next stage was to examine the behaviour of the pulse velocity measured across the compression zone of a beam subjected to flexure. A reinforced concrete beam was tested under increasing moment and the results again exhibit a similar effect. This is also shown in Figure 2 which indicates that microcracking effects may be significant at a lower relative stress level in a member subjected to flexural compression than where direct compression is involved.

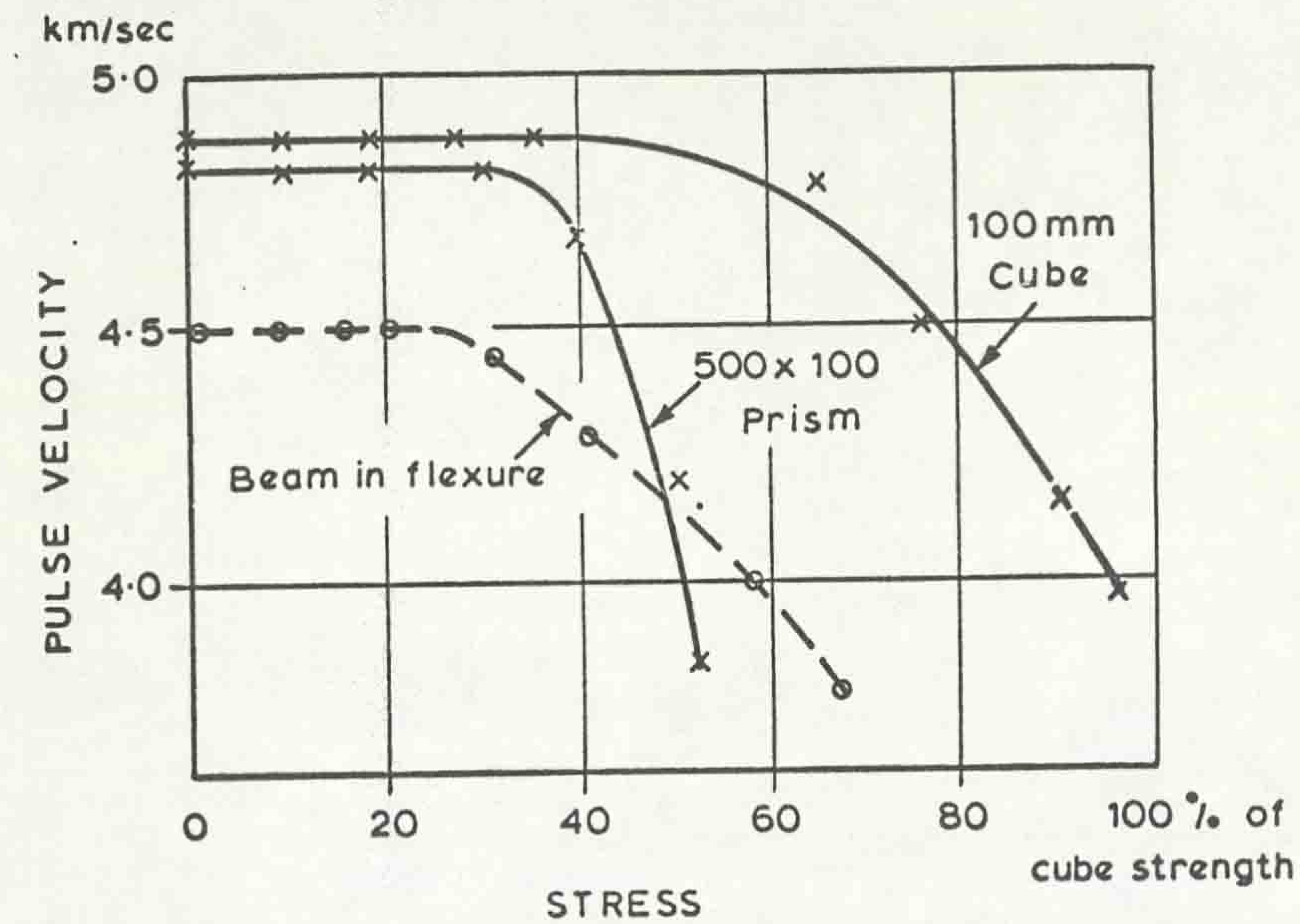

Figure 2 - Effects of Stress on Pulse Velocity

The results confirm that in stressed members, concrete strength affects pulse velocities not only in terms of the relationship with modulus of elasticity, but also in terms of the ratio of working stress to ultimate material strength. In prestressed concrete beams, these factors combine to cause pulse velocities in understrength concrete to be substantially lower than in sound unstressed laboratory specimens of a similar mix. It is this feature which enables ultrasonic testing to detect seriously substandard concrete.

Ultrasonic Testing of Small Sections

When dealing with small size sections, consideration must be given to the effects of path length and path width on pulse velocity readings. Figure 3 illustrates typical results obtained on unstressed prisms with 20mm. maximum aggregate size which were cut successively into a series of slices, thus reducing the path length for longitudinal pulse velocity readings, and the path width for transverse readings. 50kHz. transducers were used, and these results demonstrate clearly the need for a minimum path length of 100mm. for this aggregate size and fequency. Path width is not so crucial although readings tend to become increasingly erratic as the width reduces.

Where the specimen width is less than that of the transducers, it is difficult to decide between the effects of

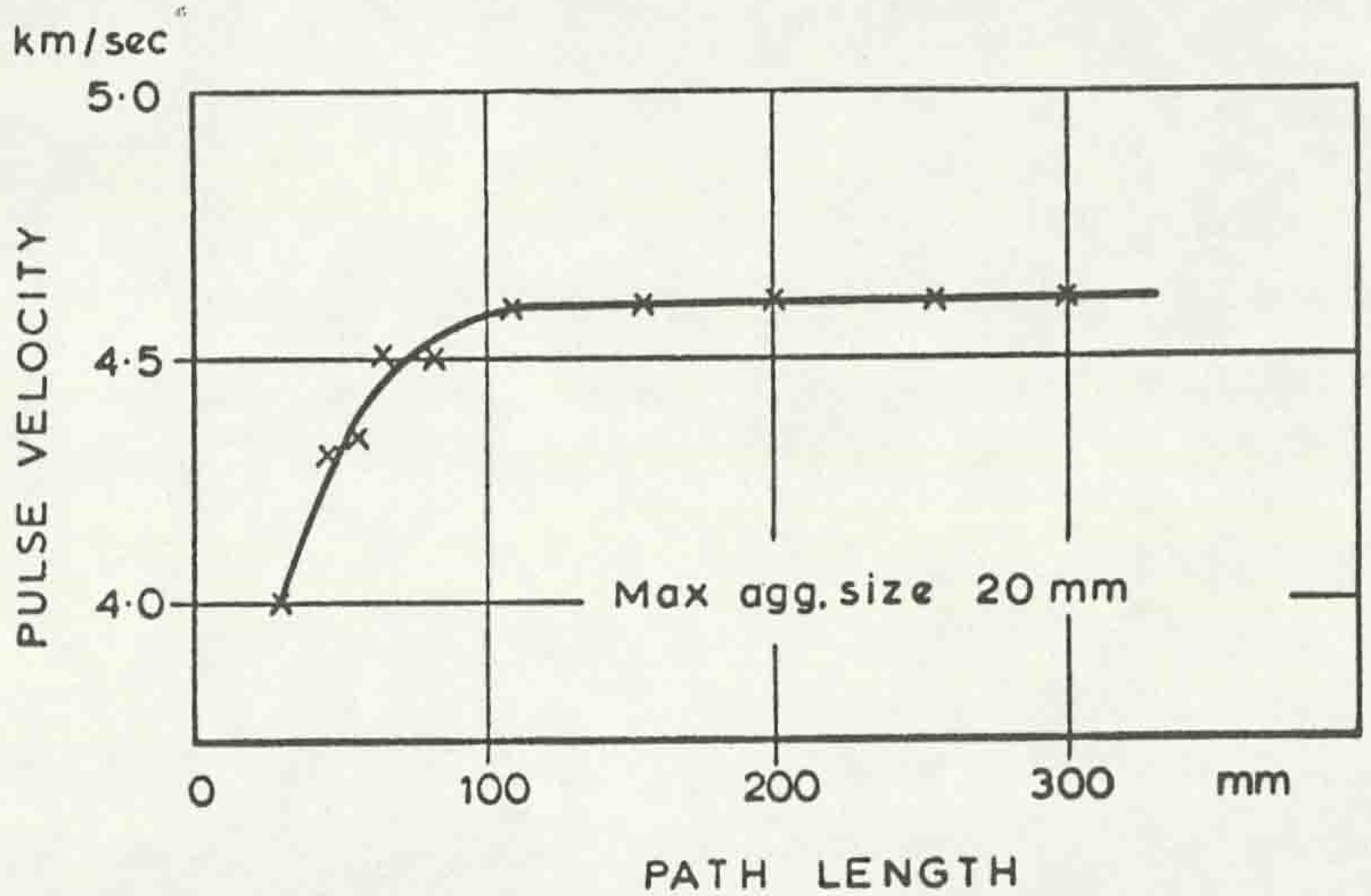

Figure 3 - Effect of Path Length on Pulse Velocity

path width and possible transducer seating problems. To examine this, a series of sections of a range of strengths were cast with dimensions typical of many small section pre-tensioned members. These were cast with control cubes and prisms upon which comparative pulse velocity readings were taken. When taking readings acorss the flanges of these sections, the transducers overhang but the path width is difficult to assess. Typical results, as in Table 1, show that there is no significant difference between readings across the cubes and across the flanges of the sections. The greater variability of results when dealing with low strength concrete should be noted. Path width, and transducer overhang do not

Mix	Average Cube Strength N/mm^2	Average Pulse Velocities Km/Sec			
		Prism	Cube	Small Beam Section	
				Flange 1	Flange 2
1	77	5.03	4.94	5.00	5.00
2	35	4.51	4.58	4.50	4.59
3	17	3.98	4.22	4.21	4.12

Table 1 - Pulse Velocities on Unstressed Specimens

appear to be critical in such cases. The decline in pulse velocity on very narrow paths can thus be attributed directly to disruption of the pulses by the narrow path.

<u>Core Testing</u>
The only method of obtaining quantitative information from pulse velocity is by calibrating, either with respect to cores or to flexural compressive strength calculated from a test to descruction of a similar beam. The latter alternative is seldom possible, since calibration must normally require a number of results covering a wide strength range. Cores, therefore, appear to offer the best hope, both of obtaining semi-destructively an estimate of concrete strength, and of calibrating pulse velocities. On large section members, 75mm. or 100mm. cores can be cut, tested and corrected in accordance with BS.1881 (4). Correlations between core strengths and calculated flexural strengths for prestressed beams have been demonstrated by Cusens and Jackson (5).

On small section members, very much smaller core diameters must be used. This introduces further problems in terms of stress distributions in a small specimen under test, and also the large size of aggregates relative to the overall specimen size. However, tests on 44mm. diameter cores indicate that accuracy of prediction is comparable although the length/diameter ratio has greater influence. Figure 6 shows results of typical 44mm. cores from prestressed beams related to flexural strengths.

ULTIMATE LOAD TESTING

Bending tests to destruction have been carried out on a wide variety of pretensioned member types and all follow the same fundamental pattern. These members, which had been removed from suspect structures, were subjected to third point loading, usually over a 4m. span. Initial behaviour is almost invariably elastic up to a point just before first visible flexural cracking of the concrete. This is followed by a region in which curvatures increase more rapidly due to the reduced stiffness of the member, and then eventually the steel yields, provided the concrete has not in the meantime failed in compression. Eventual failure occured in all the cases tested by crushing of the concrete at the top of the section, except on short spans where shear failure occurred. The effect of reduced concrete strength is therefore exhibited principally as a reduction in Ultimate Moment of Resistance. This is demonstrated by Figure 4 where the laboratory results for two beams of the same type but with considerably different estimated concrete strengths are compared. It will be noted from this that the effect of concrete strength on uncracked stiffness, and thus by implication on Modulus of Elasticity, is relatively small. The reduction in Modulus of Elasticity would appear to be only about 25%, compared with a reduction in concrete strength of 63% and a reduction in member flexural strength of 40%. The significance of this with respect to assessment methods based on measuring the stiffness or Modulus of Elasticity is self-evident.

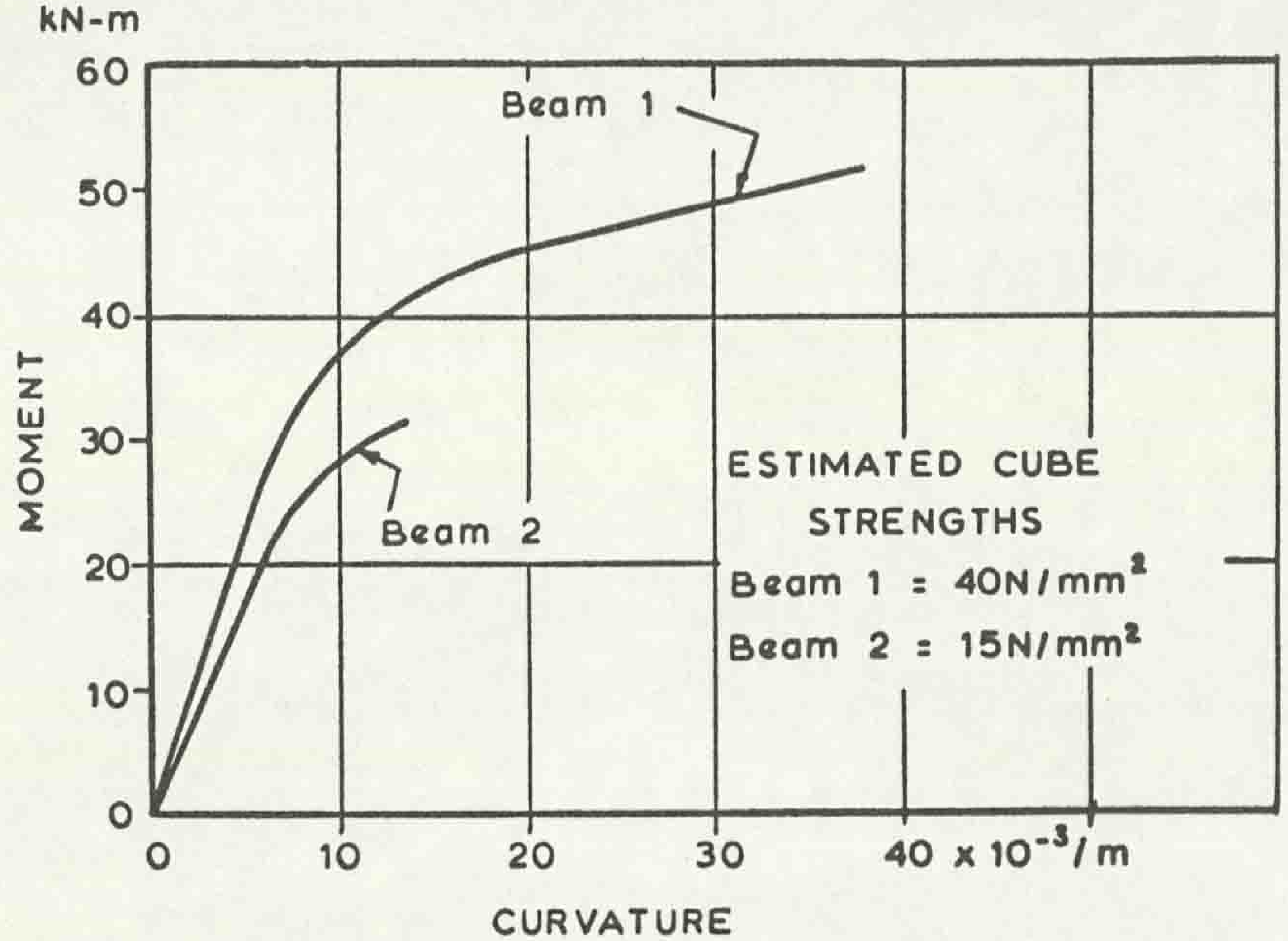

Figure 4 - Effect of Concrete Strength on Similar Prestressed Sections

The general relationship between concrete strength and member flexural strength has been discussed by a number of authors (5, 6), and Figure 5 shows theoretical curves based on a simplified rectangular stress block for two 'Pierhead' X7 type sections. The difference in behaviour between highly stressed and lightly stressed sections is evident, resulting respectively in over-reinforced and under-reinforced types of ultimate failure at normal concrete strengths. It is probable that these curves underestimate the true flexural strength for a given characteristic concrete cube strength since they are based on a characteristic prestressing wire strength of $1540 N/mm^2$ and the conservative simplified stress block. Tests on wires removed from beams indicate mean values on average 8% higher. Calculated characteristic cube strengths from moments sustained in bending tests, are compared with averages of small diameter core strengths for these same members in Figure 6. The average concrete stress in the compression zone has been taken as 0.67 x characteristic.

Figure 6 also shows estimated concrete strengths for typical members based on average pulse velocities across flanges, using calibration charts based on cores, compared with estimated strengths from flexural tests on the same members. This demonstrates that pulse velocity testing can yield a realistic estimate of member flexural strength.

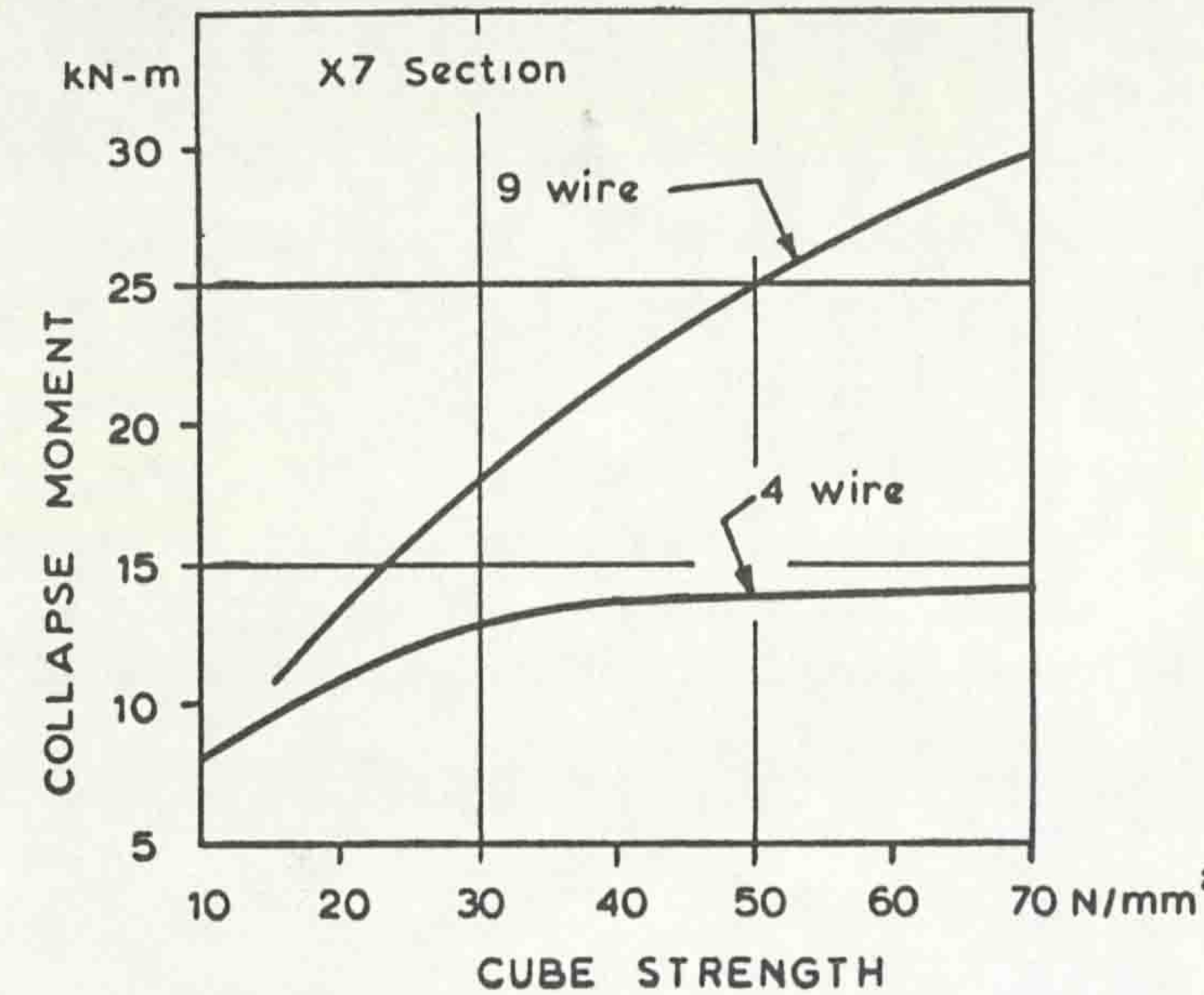

Figure 5 - Effect of Prestress on Member Strength

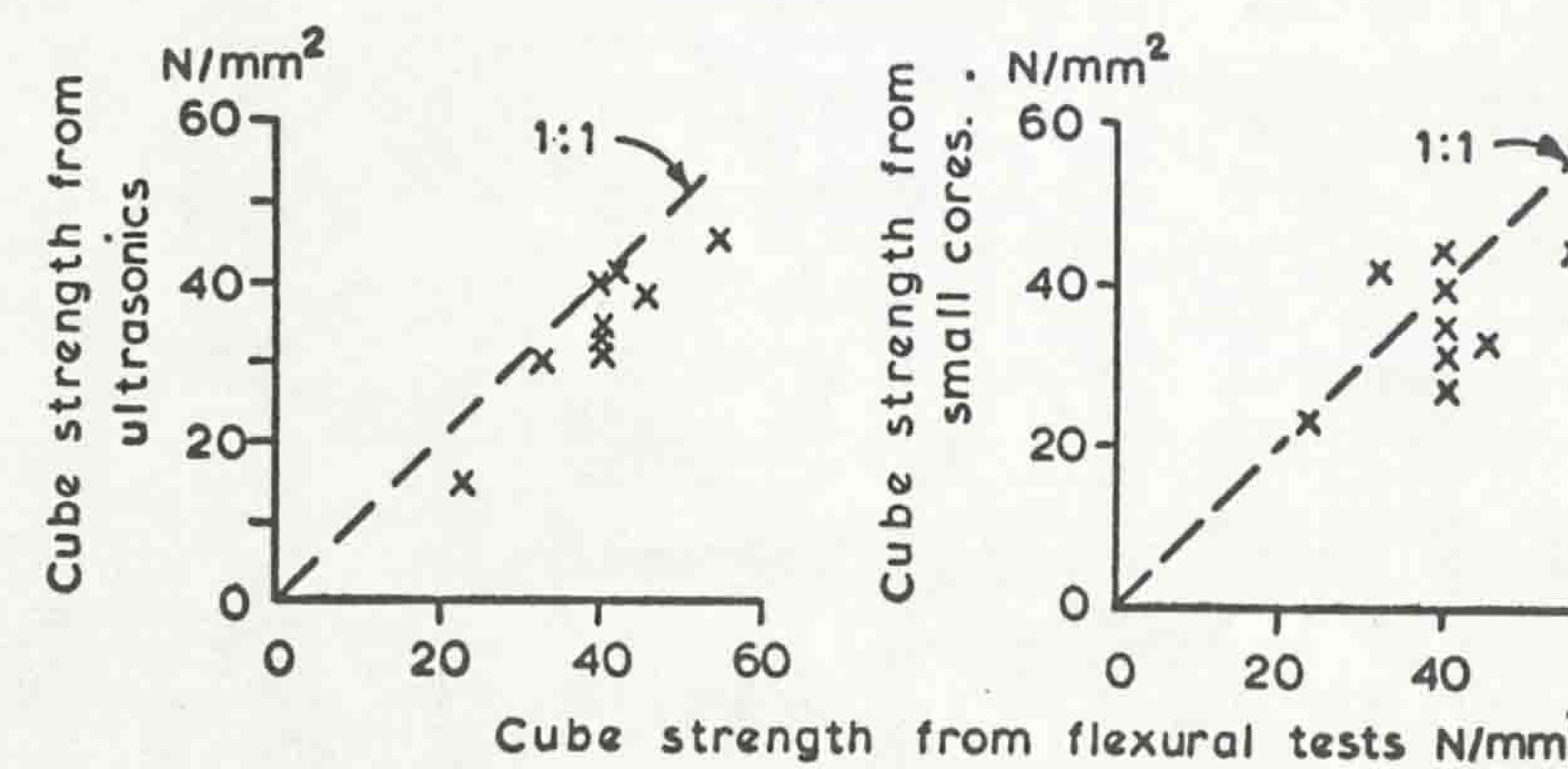

Figure 6 - Comparison of Estimated Concrete Strengths

When attempting to predict the ultimate strengths of members, however, it has been found that construction dimensional variations can in some instances be considerable, thus calculations based on the 'nominal' dimensions may well involve significant errors from this cause.

PRACTICAL PROBLEMS OF INSITU TESTING

Ultrasonics

Although non-destructive with respect to the concrete, the number of cases where individual beams are accessible for direct readings across their width are relatively few, and generally confined to roofs. A great many roofs and floors have infill lightweight pots which must be broken out on opposite sides of the beam at each test point to enable a transverse flange reading. Not only is this a messy operation which causes considerable damage to the ceilings, which are usually plastered, but it must be performed with great care if the edges of the beam are not to be damaged, thus rendering an effective reading impossible. The possible presence of flexural cracking and the presence of longitudinal prestress wires means that longitudinal indirect readings taken on the soffit of the concrete are of little value. On most sections, the flange width is usually insufficient to enable reliable transverse indirect readings. Thus when the situation occurs where beams are located as pairs which are next to each other, ultrasonics cannot realistically be used. Attempts to take readings vertically through the webs are usually frustrated by the need to remove the screed or slab above the beam, and the difficulties involved with obtaining a good transducer seating at the top.

Core Testing

The cutting of cores insitu is a messy business and requires considerable expertise. Coupled with this is the problem that although the hole can be plugged with concrete, the true effects of cutting holes in prestressed beams are not known. The use of this test would appear to be limited to large members where the effects of the cutting will be relatively small, and a reasonable diameter core may be used. With small section members, small diameter cores only are possible to maintain a minimum length/diameter ratio of 1. Core tests will probably only be used if other methods are very much more difficult, or if calibration is required for comparative ultrasonic results. If cores cannot be avoided, the most effective method of obtaining them has been to drill down into the member from above, although the member must always be propped as a safety precaution.

Load Testing

Apart from the practical problems of providing the load and measuring the deflections, the major problem is allowing for the effects of finishes. Non-structural screeds, and pots, will have a considerable load distribution effect, which renders an assessment of the load on the member under test extremely difficult. It has been found that a considerable width of slab must be loaded for the central member to be carrying the full load, thus the choice really lies between loading an entire bay of roof or floor, or isolating one or two members completely from the surrounding structure and then loading these. Either approach is expensive and disruptive, and scaffolding must always be used to support the test area in case of a collapse.

GENERAL OBSERVATIONS

In the course of insitu investigations of floor and roofing systems incorporating pretensioned beams, a number of common features have been observed. From the design point of view it is evident that environmental conditions have frequently been overlooked in connection with the usage of High Alumina Concrete. Examples observed range from insitu usage in roofs over kitchens where average temperatures in the ceiling void have been recorded in excess of 80^{o}F, to chemical laboratories where beams run over furnaces and adjacent to fume cupboards in chemical laden atmospheres. In the latter case, the concrete was visibly disintegrating in some areas. Apart from High Alumina Concrete usage problems, some simply supported roof spans have been found with span/depth ratio in excess of 50. Not surprisingly, such spans are sagging badly, despite the fact that the concrete strength is apparently acceptable. Excessive deflections are not uncommon, especially in older construction, and frequently causes considerable damage to finishes, and fittingssuch as windows.

The use of recessed ends of members at supports is not uncommon, particularly in T-section roof purlins. This type of section is inherently strong with respect to bending strength as governed by the concrete strength, but in shear they are particularly vulnerable both because of the reduced concrete section and the reduced prestress compression in the support zone. Indeed, one case has been found of a whole series of such members 'hanging' on the mild steel shear reinforcement, with wide cracks at the root of the support nib.

Timber inserts are often cast into the soffit of precast beams, to enable ceilings to be fixed. Figure 7 shows a typical example of this, and although no cases have been found of failure which can be attributed directly to this, in many cases the central wire is barely in contact with any concrete, severely corroded, and probably of little structural value. It has also been found in a number of beams with this feature that there is a tendency for part of the lower flange to split off along this line of weakness.

Construction deficiencies, in the form of bad compaction and damage to members, usually the top flanges, are fairly common. Apart from the obvious question of why the more extreme examples of these faults were accepted during

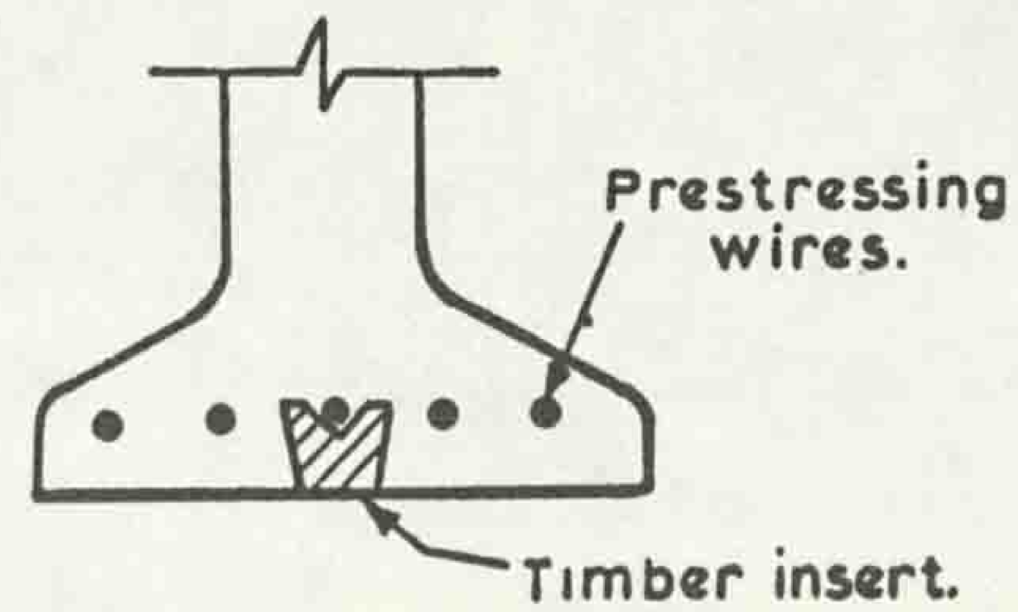

Figure 7 - Timber Insert in Soffit

construction, it is not uncommon to find that subsequent service work has sometimes involved the removal of quite large areas of concrete from the pretensioned beams. Whilst it may perhaps be argued that such faults on the odd beam amongst many are unlikely to have serious consequences, occasionally more fundamental constructional deficiences exist which change the structural action of the members, and this is potentially more dangerous. One example of this encountered in a recent investigation is the case of a warehouse roof which was supported on 7" deep X section joists running between large rectangular section main beams. As can be seen from the support detail in Figure 8, a certain amount of end continuity was provided by the design. In practice it was found that not only were many of the seatings reduced to as little as 30mm. with spalling visible on the main beams beneath, but in some entire bays the infill concrete was missing completely. Nowhere in the entire structure was the joint constructed as detailed. Structural collapses seldom occur due to one cause only, but fundamental faults such as this produce a situation where the structure is vulnerable, and some other relatively minor factor such as chemical attack from a leaky roof, or strength loss from another cause may be sufficient to cause collapse.

Maintenance, particularly of roofs, is of utmost importance. Leakage problems, especially of flat roofs, are well known and yet it is relatively few roofs that on inspection are found to be completely dry. In addition to damage to interior finishes, the water may transport chemicals leached from the roofing material, and if a steady supply reaches the surface of the concrete members, attack may well occur. This of course is especially important where High Alumina Concrete is concerned, because of the very low resistance of the converted concrete to sulphates and other common chemicals used in roofing materials and plasters. The problem is again not confined to High Alumina Concrete, but if the concrete is porous, water may be absorbed and lead to corrosion of prestressing wires and reinforcement. Apart from the potential loss of strength resulting from this, the protective cover concrete will tend to crack and spall off, thus aggravating the situation.

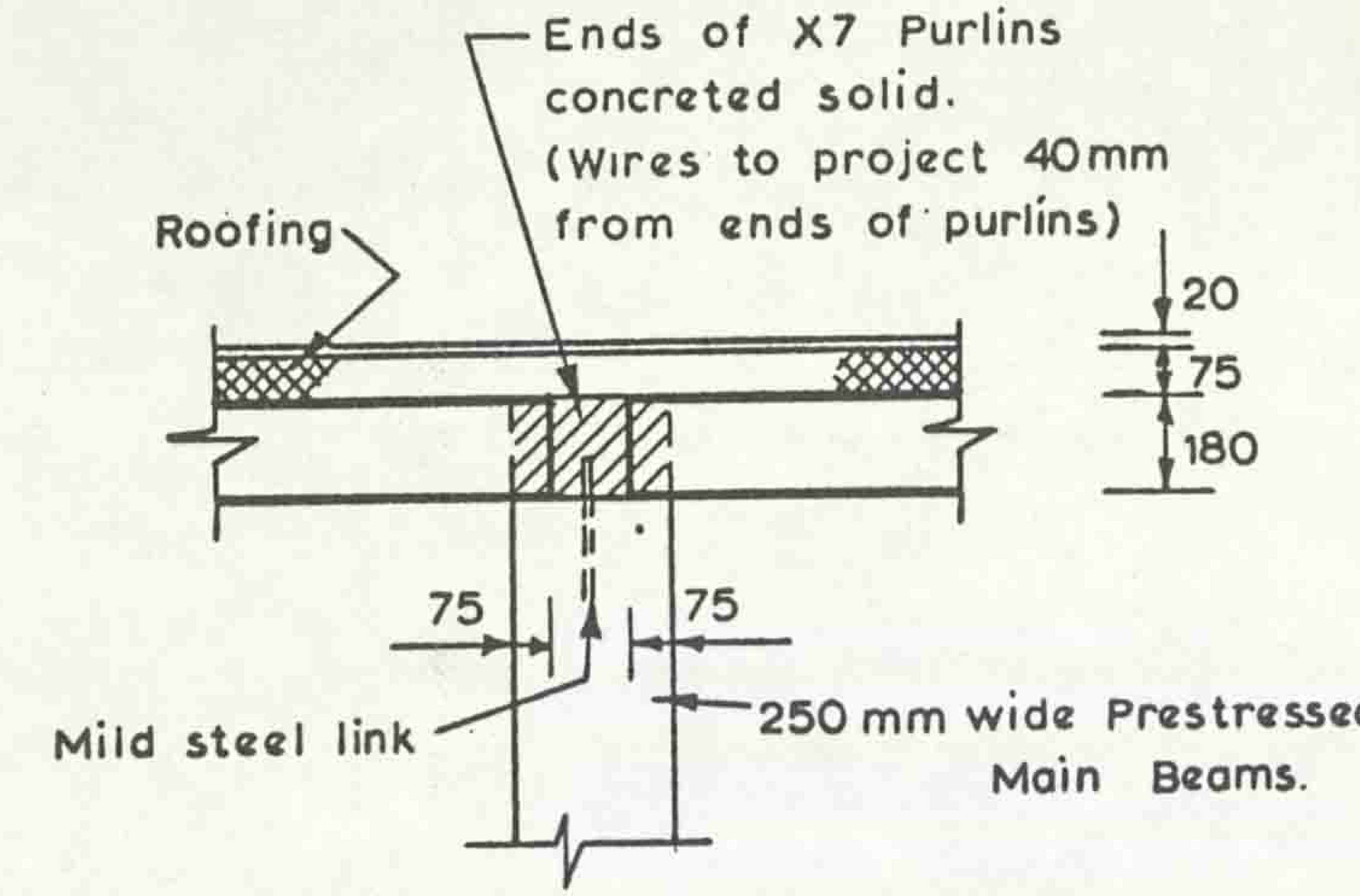

Figure 8 - Purlin Support Detail

CONCLUSIONS

The engineer attempting to assess the structural condition of a roof or floor which incorporates prestressed concrete can seldom hope to find an easy solution. Considerable expense in terms of disruption of usage, preparatory work, testing and making good is inevitable. With the exception of carefully executed insitu load tests where a specified margin of safety is demonstrated at a given time, recourse must be made to the results of experimental work and previous similar field investigations when attempting to interpret test results. In the case of a situation which is not stable with respect to time, these problems are greatly aggravated.

The results of the laboratory ultimate bending tests reported here indicate that it is possible to predict the behaviour of individual small section members with a reasonable degree of certainty on the basis of ultrasonic and core tests coupled with simple conventional theory. The extension of this information to the likely behaviour of such members under insitu conditions must however remain a matter of engineering judgement at the present time, especially where composite action is involved.

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Paper 7

"The validity of ultrasonic pulse velocity testing of in-place concrete for strength"

NDT International Dec. 1980 pp.296-300

The validity of ultrasonic pulse velocity testing of in-place concrete for strength

J.H. Bungey

It is generally accepted that the use of ultrasonic methods for assessment of concrete suffers many disadvantages, particularly where a strength estimation is required. The absence of alternative non-destructive methods capable of giving significantly better results, however, means that despite these difficulties ultrasonics have a valuable role to play, especially in the field of in-place evaluation of structural concrete. It is thus worthwhile to examine in some detail those factors having a major influence on 'field' results, with the aim of assessing the confidence with which they may be viewed.

Materials and calibration problems

The basic problem is that the material under test consists of two separate constituents, matrix and aggregate, which both have separate elastic and strength properties. The relationship between elastic modulus and strength of the composite material cannot be defined simply by consideration of the individual constituent properties in relation to their proportions. This is because of the influence of aggregate particle shape, efficiency of the aggregate/matrix interface and variability of particle distribution, coupled with changes of matrix properties with age. Although some attempts have been made to represent this theoretically, the complexity of the inter-relationships is such that experimental calibration for elastic modulus and pulse velocity/strength relationships is normally necessary. Aggregate may vary in type, shape, size and quantity, while the cement type, sand type, water/cement ratio and maturity are all major factors which influence the matrix properties. Hence a pulse velocity/strength curve obtained with maturity as the only variable will differ from that obtained by varying the water/cement ratio for otherwise similar mixes, but testing at comparable maturities.

The effect of moisture condition on both pulse velocity and concrete strength is a further contributory factor to calibration difficulties since the moisture content of concrete will generally decrease with age. A moist specimen shows a higher pulse velocity, but lower measured strength than a comparable dry specimen, thus drying out results in a decrease in measured pulse velocity relative to strength. This effect is well illustrated by the results in Fig. 1 which relate to otherwise identical laboratory specimens, and demonstrates the need to correlate test cube moisture to structure moisture during strength calibration. It is thus apparent that strength correlation curves are of limited value for application to in-place concrete unless based on the appropriate maturity. Tomsett[1] has recently presented an approach which permits calibration for 'actual' in-situ concrete strength to be obtained from a correlation based on standard control specimens. This simple approach allows for both strength and moisture differences between insitu concrete and control specimens and may prove to be valuable. However, a direct strength assessment of a typical reference specimen of in-situ concrete is still required if the relationship is to be used for other than comparative applications.

Testing technique

If genuine variations in pulse velocity are to be detected it is essential that considerable attention is paid to preparation of the concrete surface to ensure good contact between the transducers and the concrete member. Surfaces cast against steel or good quality plywood moulds will generally be satis-

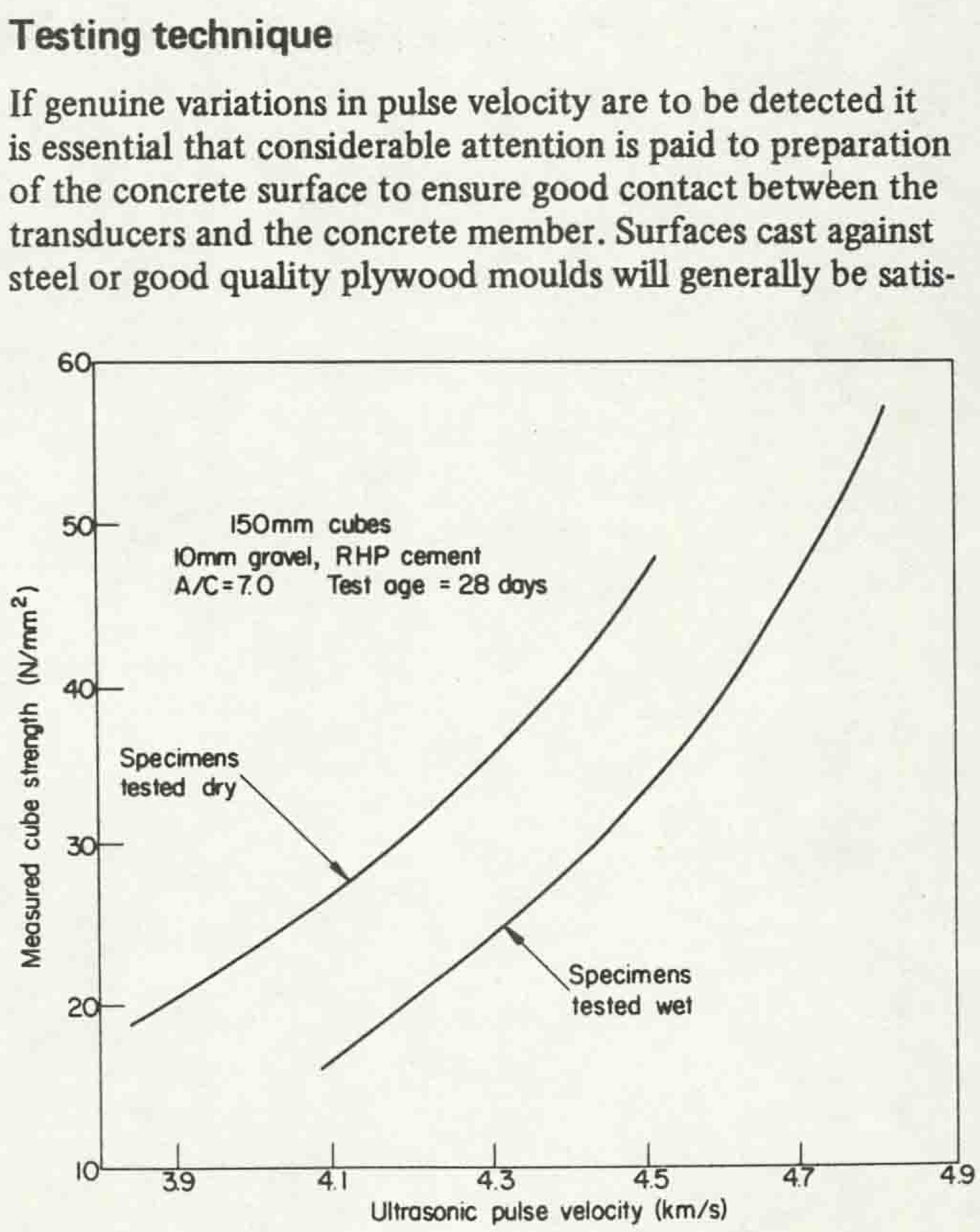

Fig. 1 The influence of moisture conditions on pulse velocity

factory except in the region of shuttering joints, but all other surfaces must be ground smooth. Choice and application of couplant is also important, although for sound smooth surfaces petroleum jelly is often considered to give the best results. The most common type of transducer has a flat surface diameter of 50mm and therefore good contact must be ensured over a considerable area. However, the use of a probe transducer making only point contact and normally requiring no surface treatment or couplant offers advantages. Time savings may be considerable and path length accuracy for indirect readings may be increased, but this type of transducer is unfortunately more sensitive to operator pressure. Receivers have been found to operate satisfactorily in the field, but the signal power available from a transmitting transducer of this type is so low that its use is not normally practicable for site testing.

There is little doubt that much of the scepticism of engineers towards ultrasonics results from testing that has not been carried out by skilled personnel who are fully aware of these difficulties.

Measurement problems relating to member under test

Member size

The only major difficulties arise when small size members are involved and the medium under test cannot be considered as effectively infinite. This will occur when the path width is less than the wavelength λ. Since λ = Pulse velocity/Frequency of vibration it follows that the least lateral dimensions given in Table 1, which relate to the most commonly used transducers, should be satisfied. Aggregate size must similarly be less than λ to avoid reduction of wave energy and possible loss of signal at the receiver, although this will not normally be a problem.

Although the path length is not important theoretically, physical limitations of the time measuring circuits in generally available equipment can introduce errors where short paths are used. Typical results obtained on a laboratory specimen which was incrementally reduced by sawing are shown in Fig. 2 and clearly demonstrate this effect. Aggregate size is also a factor, and the suggested limits in Table 1 are based on RILEM[2] and British Standard[3] recommendations. Maximum path length is governed by the sensitivity of the equipment in use, but suggested values applicable to the equipment most widely used in the UK (PUNDIT) are also given in Table 1. These may, however, be increased significantly by the use of an inexpensive signal amplifier. Provided that care is given to the selection of the most appropriate transducer frequency, member size is thus unlikely to cause major measurement problems, especially where full-size structural members are concerned.

Table 1. Limits to path dimensions

Frequency (kHz)	Least lateral dimension or maximum aggregate size (mm) V_c = 3.8 km/s	V_c = 4.6 km/s	Recommended Path Length Range (20 mm maximum aggregate)
54	70	85	100mm - 10m
82	46	56	100mm - 3m

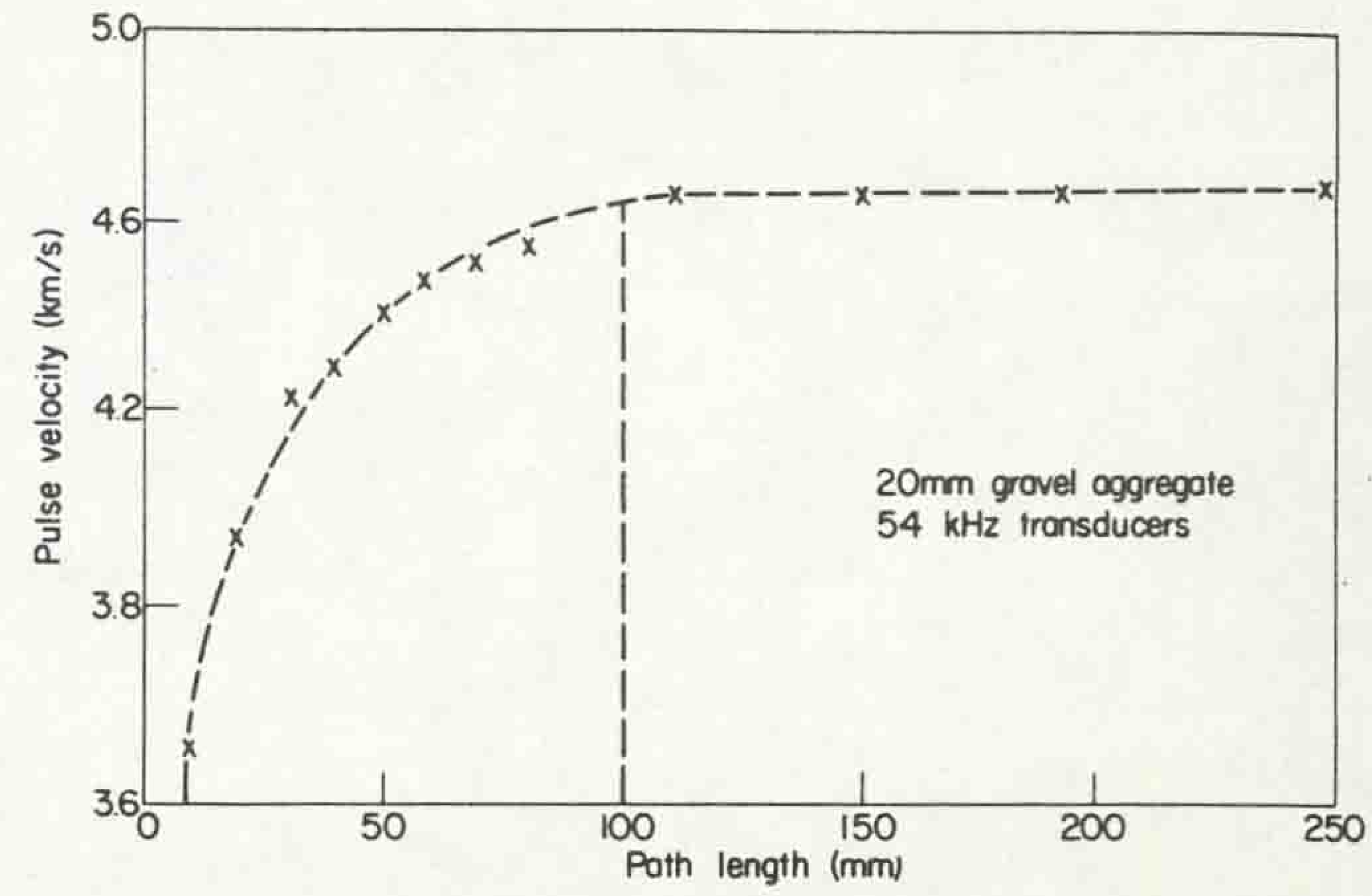

Fig. 2 The effect of short path length

Influence of Reinforcement

The pulse velocity through an infinite steel medium is generally about 5.90 km/s but decreases with diameter in the case of reinforcing bars due to inadequate path width (Fig. 3). Nevertheless, this value is considerably greater than for even good quality concrete, and it may be expected that the presence of steel reinforcement will influence results. Chung[4] has shown that for pulses travelling in the direction of the axis of reinforcing bars through a steel/concrete medium, the 'effective velocity' is considerably less than the theoretical value and is influenced by bar diameter. Chung's results give an effective measured pulse velocity:

$$V_e = 5.90 - 10.4\,(5.90 - V_c)\,/d$$

where

V_c = true velocity in concrete; and

d = diameter of bar

This is based on a 54 kHz frequency, and examination of the authors, results in Fig. 3 suggests that measured velocities will be marginally higher where a higher frequency is used. It appears, nevertheless, that the use of the reinforcement correction factors in BS 4408 Pt 5 will lead to a major understimate of the true pulse velocity of the concrete

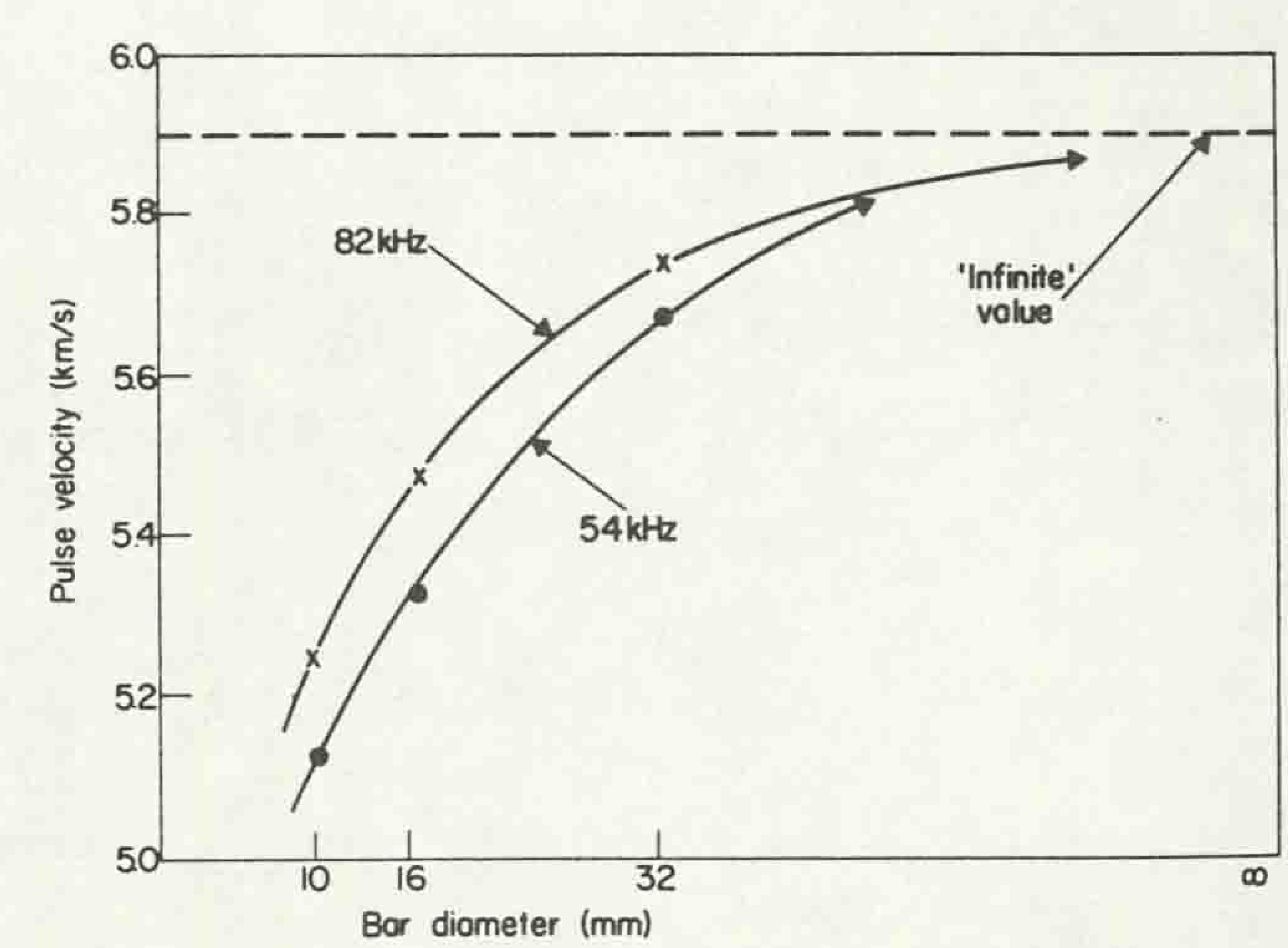

Fig. 3 Pulse velocity vs bar diameter through reinforcing bars in air

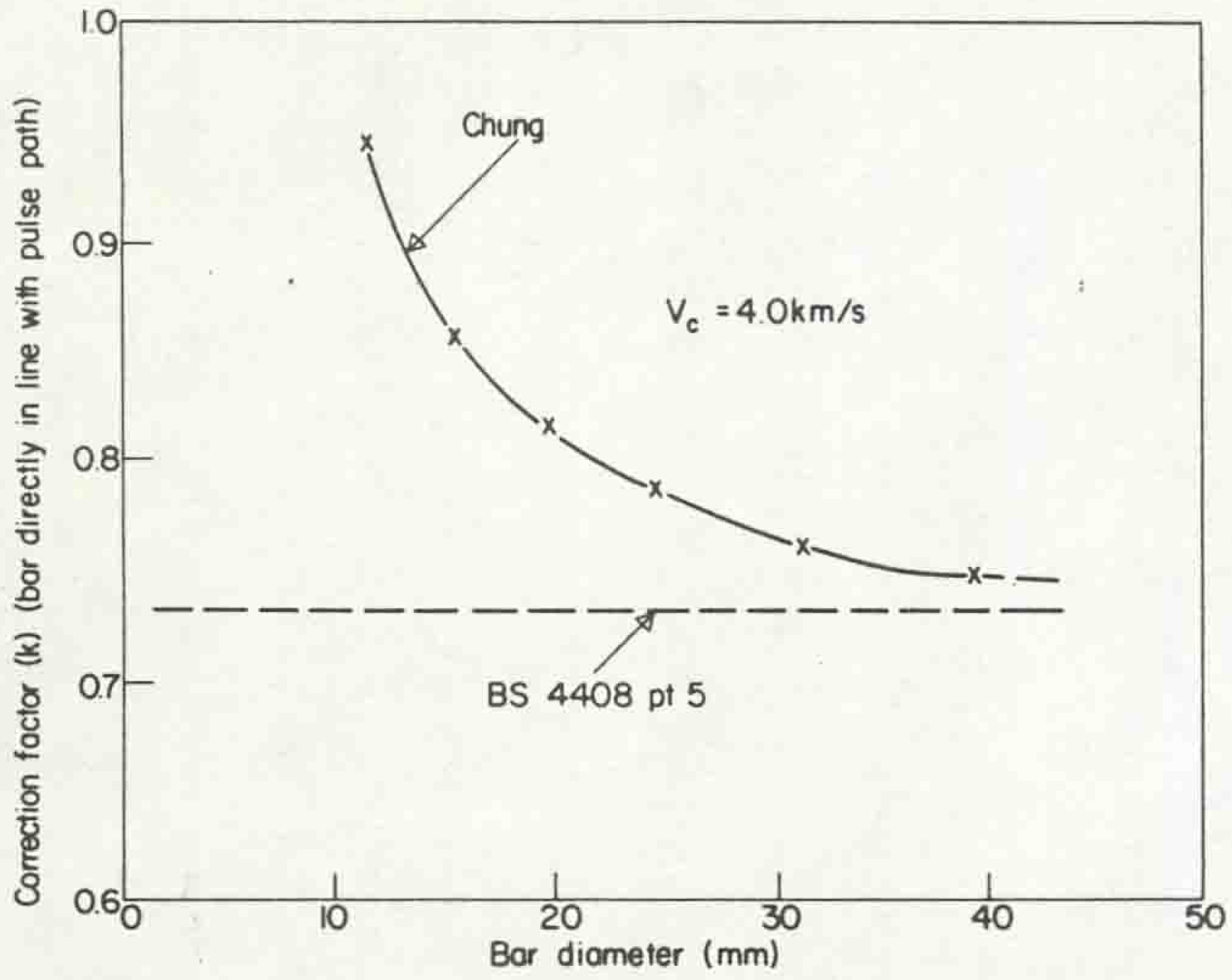

Fig. 4 The influence of bar diameter on correction factors for longitudinal reinforcement. Chung's[4] results are shown.

except where very heavy reinforcing bars are used, as illustrated by Fig. 4. This is because the standard provides 'maximum effect' correction factors which do not take any account of the reduction in 'effective' velocity with bar diameter.

Recent tests by the author have not only confirmed these findings but shown that for transverse bars the influence is even less. This is no doubt due to the very small proportion of the path length through the steel being of the full diameter. Hence, it may be possible to consider a transverse bar as being of a reduced 'effective' diameter.

Effects of Stress

It is generally accepted that for laboratory cubes under compressive stress the pulse velocity is not significantly affected until a stress of approximately 50% of the crushing strength is reached.

A series of three No 300 x 150mm beams of 3m length have been made and tested in the laboratory to examine the influence of a flexural stress condition, since this is of greater practical relevance than the complex stress conditions existing in a cube. The beams were subjected to 1/3 point loading, with 82 kHz transducers clamped in position on the opposite side faces as close to the top surface as possible prior to loading. The simple clamp used is illustrated in Fig. 5, and this approach was found to effectively eliminate measurement variability and permit continuous monitoring. Strain measurements were taken using a Demec gauge at various levels of the beam to permit stresses to be estimated on the basis of the stress-strain relationship recommended by CP110.[5] 150 mm cubes were also cast from the 10 mm gravel aggregate mixes, cured with the beams, and subjected to UPV measurements at various stress levels prior to crushing.

The results of these tests are summarised in Table 2. Two of the beams were prestressed while one was conveniently reinforced. The effects of compressive stress are expressed as the percentage of the measured cube strength of the concrete necessary to cause a 0.5% drop in pulse velocity. The results confirm the findings of other workers in relation to the behaviour of cubes and, more significantly, demonstrate that the characteristics of concrete subjected to flexure are similar. It is important to note that concrete in flexural compression fails at a stress of approximately 2/3 of the cube strength, thus the reduction in beam pulse velocity does not occur until a stress level of at least 70% of the failure value is reached. Pulse velocities were similarly monitored near the soffits of the post-tensioned beams during stressing, but no detectable changes were observed.

It was clearly shown that under service conditions in which stresses would not normally exceed 1/3 cube strength the influence of compressive stress on pulse velocity is insignificant, and that pulse velocities for prestressed concrete members may be used with confidence. Only if a member has been seriously overstressed will pulse velocities be affected. Tensile stresses have been found to have a similarly insignificant effect, but potentially cracked regions should be treated with caution, even when measurements are parallel to cracks since these may reduce path widths below acceptable limits.

Interpretation of measured values

The scatter of results obtained from testing members of nominally identical concrete is partially caused by testing variations but will be predominantly due to variability of the concrete. Tests on many sets of 'similar' laboratory cast test cubes have indicated an average coefficient of variation for pulse velocity of 0.5%. Under similar conditions strength/pulse velocity relationships based on 28-day results with varying water/cement ratio and almost constant aggregate cement ratios have yielded correlation coefficients of the order of 0.95. Such relationships would enable the strength of a similar cube to be estimated from pulse velocity readings with 95% confidence limits of about ± 10%. It is unlikely

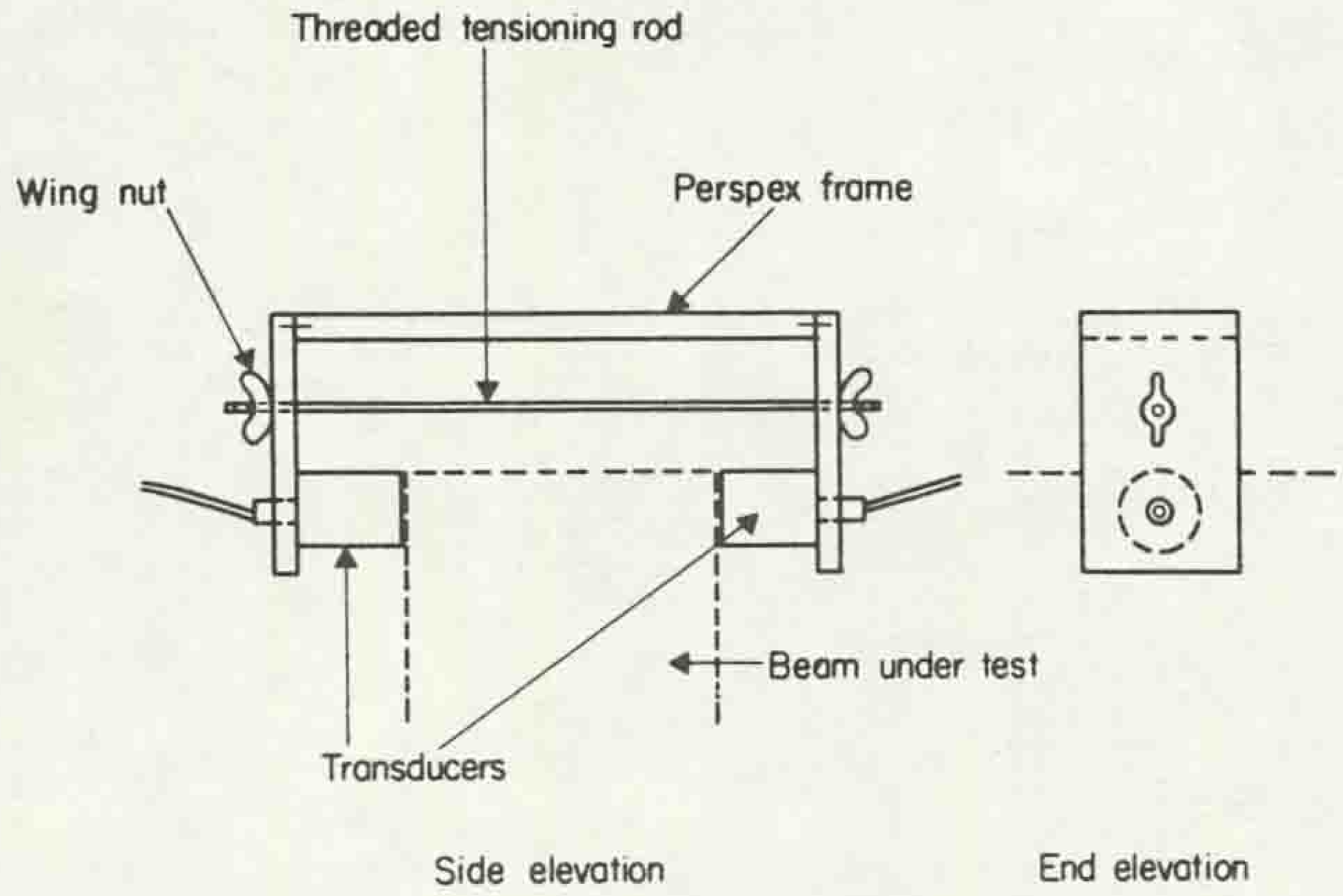

Fig. 5 A schematic view of the transducer clamps

Table 2. Influence of compressive stress

Beam reference	Type	Average measured cube strength, f_c (N/mm^2)	% Reduction in PV due to Stressing to 50% f_c	Stress for 0.5% reduction in PV (as % of f_c)	
				Cube	Beam
A	RC	44	-	65	60
B	PSC	35	0	50	60
C	PSC	20	0	45	48

that the testing variabilities will be reduced significantly below this figure, especially for site work.

The variability of concrete in structural members is also such that the reliability of overall strength prediction is greatly reduced in comparison with ideal conditions. This will be due to variations both in concrete supply and the efficiency of compaction, and will be greatly influenced by the construction conditions. Tomsett[1] suggests that for a single site made unit constructed from a single load of concrete a pulse velocity coefficient of variation of 1.5% would reflect good construction, increasing to 2.5% if several loads are involved. A corresponding typical value of 6-9% is also suggested for a whole structure. These values may be compared with results reported by the American National Bureau of Standards[6] which show an average of 8% coefficient of variation for tests on slabs between one and 14 days old using a ready-mixed concrete. For this concrete, the strength calibration relationship based on varying age yielded a correlation coefficient of only 0.87. It may, therefore, reasonably be assumed that the high value of coefficient of variation obtained for a single member reflects the variable material suggested by the low correlation coefficient, coupled with variations in rates of drying of the slab. Compaction is likely to be more uniform for a slab than for a beam or column.

The results of recent tests on two 0.5m deep reinforced concrete beams cast under laboratory conditions from a number of batches are illustrated in Fig. 6. These beams were allowed to dry out under warm laboratory conditions for several months prior to test, thereby minimising moisture effects. Pulse velocity contours for Beam 1 show the effects of variable compaction coupled with the general trend of reduction in strength from bottom to top which is common in large concrete members. Beam 2 shows a greater uniformity of compaction along its length, but a 'rogue' batch is suggested by the high values at the left hand end. This was confirmed by an unusually high control cube strength for this batch. The overall results are summarised in Table 3 and agree well with Tomsett's suggestions[1]. It is worth noting that if the 'rogue' batch is discounted, the overall coefficient of variation of pulse velocity for Beam 2 is similar to Beam 1, and that at any particular level of the beams the average coefficient of variation of pulse velocity was 1.3%. The subsequent values of the coefficient of variation of concrete strength result from the direct use of an appropriate calibration curve, and may thus be regarded principally as an indication of concrete variability within the member. When the reliability of this calibration, which had a correlation coefficient of 0.97, is taken into account it can be shown that an overall strength prediction for such a member based on a mean pulse velo-

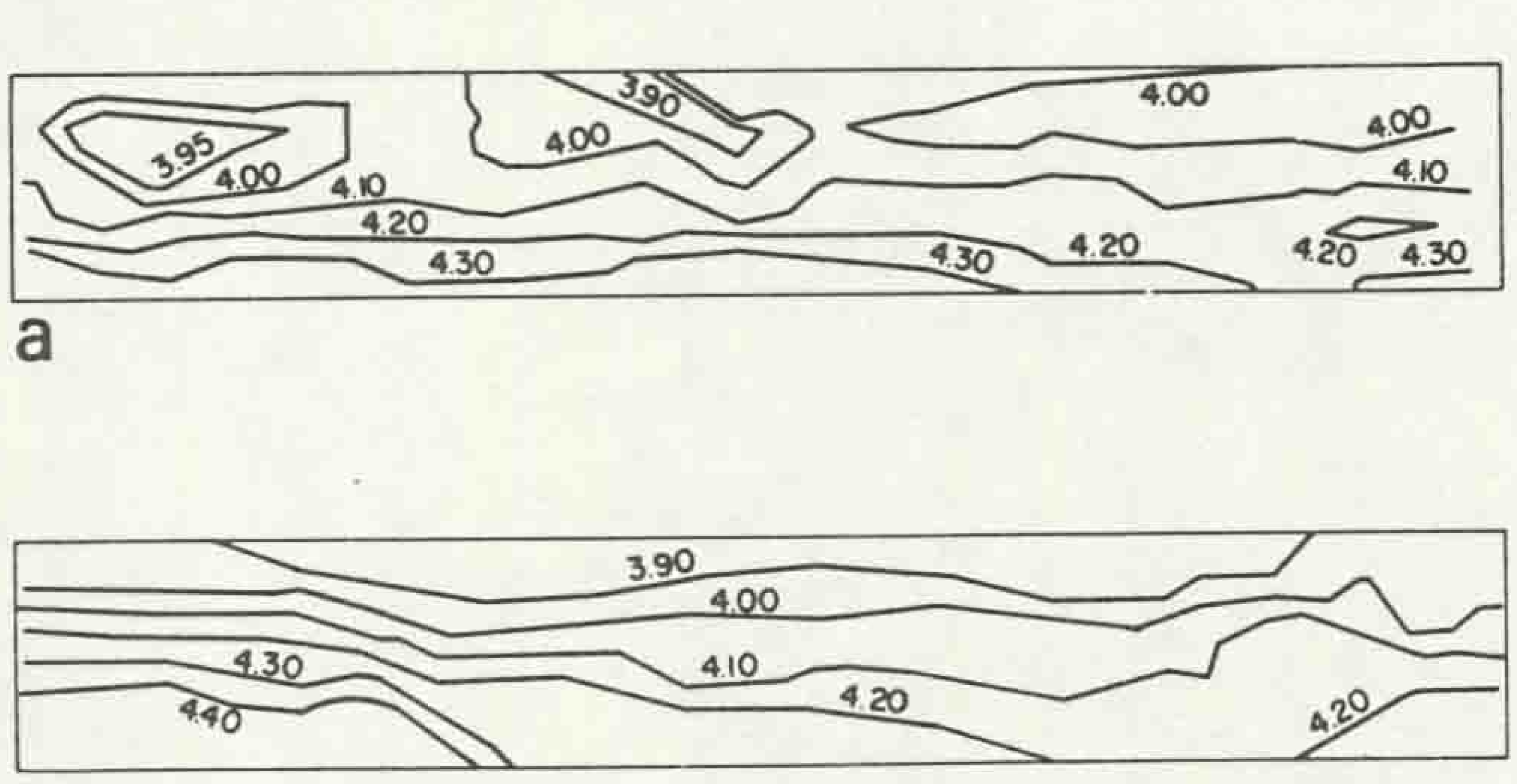

Fig. 6 The pulse velocity contours for beams in km/s. (a) Beam 1, (b) Beam 2

Table 3. Summary of beam test results

Beam	Mean pulse velocity (km/s)	Overall C of V for pulse velocity (%)	Mean cube strength (N/mm²)	Overall C of V for predicted concrete strength (%)	Overall standard deviation for predicted concrete strength (N/mm²)
1	4.06	2.5	25	14	3.6
2	4.12	4.8	29	31	9.0
2 (excluding 'rogue' batch)	4.07	2.9	26	19	5.0

city is unlikely to have a standard deviation of less than 6 N/mm² at this strength level. The corresponding 95% confidence limits of ± 10 N/mm² are unlikely to be of significant value as far as determining specification compliance is concerned. It follows, therefore, that on larger members, readings for strength assessment should be concentrated upon localised areas of interest. In such a situation it may prove possible to obtain 95% confidence limits of ± 5 N/mm² for concrete of this strength given good testing conditions and a reliable calibration curve.

Discussion

Given appropriate strength/pulse velocity calibration charts which relate both to mix characteristics and maturity of the concrete under examination, it is clear that moisture and steel present the major potential sources of error in strength assessment. Both can be adequately dealt with, but in practice moisture differences are often overlooked while steel corrections may well yield a falsely pessimistic view of the actual concrete strength. From the measurement point of view the size of the member is important in determining the most suitable pulse frequency and path location. Member type and size is also of crucial importance when interpreting results due to well-established and usual variations of the hardened concrete within the member. It is thus essential that a test programme is planned with this factor in mind in relation to the purpose and aims of the tests, especially where this is determination of strength specification compliance.

Despite the variations normally to be expected within a concrete member, an assessment of the variability of measured results can provide a valuable guide to quality, and techniques for this are well illustrated by Tomsett.[1] While relative strength comparison within a member, or between similar members can be reasonably accurate, the reliability of absolute strength prediction of a body of concrete is poor and even under ideal conditions is unlikely to achieve 95% confidence limits better than ±20%. This is appreciably worse than the accuracy to be expected from cores cut from the concrete, unless these are of particularly small diameter.[7] Nevertheless, the great advantage of ultrasonic testing of concrete is that it is the only truly non-destructive test method which enables the interior of a concrete member to be examined. Although the assessment is not based on a direct measurement of strength, there is little doubt that in many situations this approach may provide the only feasible method of strength estimation, even if this is imperfect.

Conclusions

The following conclusions may be drawn in relation to the strength assessment of in-place concrete:

A detailed knowledge of the relative moisture conditions under test is vital in establishing a reliable actual strength correlation. Failure to take account of this is most likely to cause an underestimate of in-place strength, and this error may be substantial.

Strength calibrations must be produced in a manner appropriate to the application and must be for specific aggregate and cement types. Aggregate proportions, water/cement ratio, and age must all be regarded as variables, and a pulse velocity/strength relationship must be based on laboratory specimens for which two of these are similar to the in-place concrete. The most common practical situation will involve mixes with constant aggregate/cement ratios and age at test, since strength is most affected by changes in water/cement ratio. Monitoring of early age development, however, may require calibration for a constant water/cement ratio. Unless such procedures are adopted, little confidence can be placed on in-place strength predictions.

Member or aggregate size is unlikely to affect measurements in most practical situations, but may influence the selection of test equipment, especially where small members are involved.

Past or present concrete stress conditions are unlikely to affect readings on members in either flexure or direct stress unless the concrete is seriously over-stressed.

Reinforcement effects are likely to be over-estimated using recognised approaches which do not account for bar diameter, possibly resulting in a serious under-estimate of concrete strength. It is suggested that bars of 12 mm diameter or below may be ignored, whilst Reference 4 provides a guide to the influence of larger bar sizes.

Ultrasonic pulse velocity testing provides an excellent method of examining the distribution of strength of concrete within a member, and analysis of test result variability permits a measure of construction quality.

The location of test positions must be determined to take account of likely concrete strength variability within full size members. As a general rule, strength decreases from bottom to top according to the nature of the member and the construction techniques involved.

An absolute strength prediction for in-place concrete is unlikely to have 95% confidence limits of better than ±20%, even under good conditions of testing and calibration.

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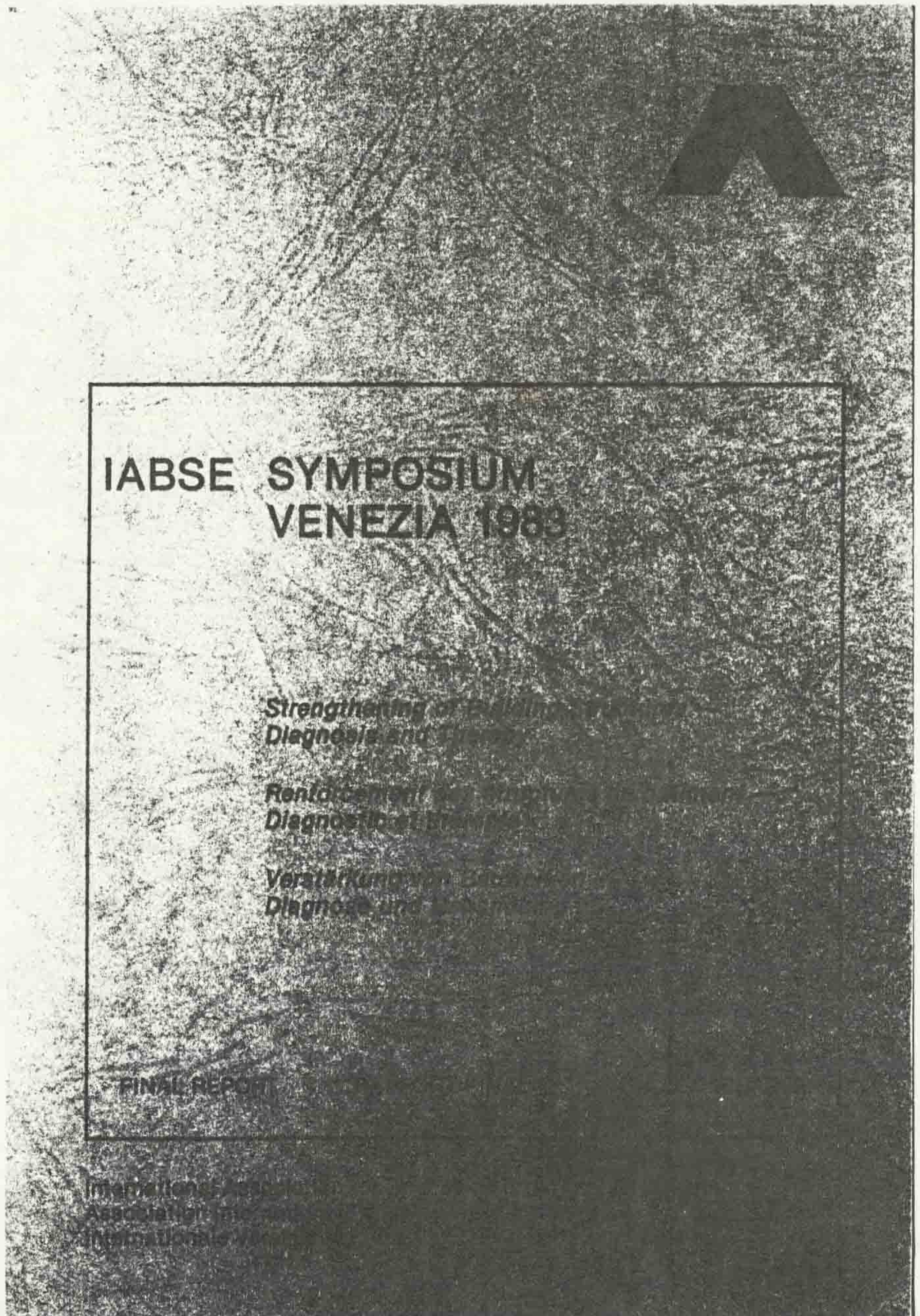
Paper 8

"Effects of Steel on Ultrasonic

Measurements for Concrete Members"

IABSE Venice Symposium 1983



Effects of Steel on Ultrasonic Measurements for Concrete Members

Effets de l'acier sur les mesures par ultrasons dans les éléments en béton

Einfluss der Bewehrung auf Ultraschallmessungen an Betonelementen

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SUMMARY

Embedded reinforcement may have a significant effect on ultrasonic pulse velocity measurements taken through concrete members. If test locations cannot avoid the influence of reinforcement it is essential that reliable corrections be made. This paper demonstrates that currently accepted allowances are unsatisfactory in practice and confirms that bar diameter is an essential variable to be considered. A correction procedure is proposed which has been developed from the results of laboratory testing.

RESUME

Les barres d'armature peuvent avoir un effet important sur les mesures de vitesse de pulsation ultrasonique au travers d'éléments en béton. Si les emplacements de mesure ne peuvent pas éviter l'influence de l'armature, il est de toute nécessité de faire des corrections sûres. Cet étude démontre que les rectifications couramment acceptées ne sont pas satisfaisantes en pratique et confirme que le diamètre de la barre est une variable essentielle dont il faut tenir compte. Un procédé correctif a été développé à partir des résultats d'expériences.

ZUSAMMENFASSUNG

Eingebettete Bewehrung kann einen grossen Einfluss auf die Ultraschallmessungen an Betonelementen haben. Wenn mittels der Messposition der Einfluss der Bewehrung nicht vermieden werden kann, ist es notwendig, Korrekturen einzuführen. Der Beitrag zeigt, dass die zur Zeit in der Praxis akzeptierten Korrekturfaktoren nicht befriedigend sind und bestätigt, dass der Durchmesser der Bewehrung eine wesentliche Variable ist. Ein aus den Laborversuchen entwickeltes Korrekturverfahren wird vorgeschlagen.

1. INTRODUCTION

1.1 Significance of Reinforcement

It is well established that embedded reinforcement which is located along, or close to, the line of ultrasonic pulse velocity measurements will influence the measured values. This is recognised by National and International Standards which recommend that reinforcement should be avoided whenever possible when selecting test locations. Such an approach is clearly the most reliable. With the aid of cover measurement devices this may often be practicable, but there will also be circumstances in which this proves to be impossible. In these cases it is necessary to make a correction to the measured value to provide an estimate of the velocity of the pulse in the plain concrete. Corrections of this type are not easy to establish because of the nature of the variables involved, and the steel influence may dominate over the concrete properties. This will inevitably reduce the confidence that can be placed in the value obtained, but careful examination of the parameters involved may help to reduce the uncertainty.

1.2 Existing Allowances and their Shortcomings

The current recommendations given by British Standards [1] and RILEM [2] for this are essentially similar and involve only the two basic parameters of concrete pulse velocity and relative pulse path lengths within the steel and concrete. An average pulse velocity through embedded steel (greater than through concrete) is assumed, and factors are given to allow for the maximum possible influence of the steel. In practice it has been suggested [3] that the diameter has a considerable effect on the pulse velocity within the steel bar. This value is further affected by the velocity of a pulse through the concrete surrounding the bar, and the condition of the bond between steel and concrete may also be important. The presence of cracking in the concrete will further complicate the situation.

The use of correction factors which do not allow for these features may result in a significant underestimate of the true pulse velocity within the concrete and lead to subsequent misinterpretation of the test results. This is a major shortcoming of the currently recommended allowances which only provide an indication of the maximum possible effects of reinforcement. These are of limited value, and may be very misleading for practical situations with common bar sizes.

1.3 Aims of Investigation

The results presented in this paper have been obtained in the course of a laboratory investigation to examine the influence of a range of variables upon the effects of embedded steel on pulse velocity measurements. The purpose of the work was to confirm and extend the basic findings of Chung [3] in relation to the currently recommended corrections, and to consider the significance of bond defects and cracking. More realistic correction procedures can thus be identified and their reliability assessed.

2. THEORETICAL BACKGROUND

2.1 Basic Theory

The influence of steel may be of importance whenever it is possible for a pulse to arrive more quickly at the receiving transducer by taking a path passing partly through the steel rather than through the concrete alone. The important features are therefore:-

a) location of reinforcement relative to the transducer positions.
b) pulse velocity within the concrete (V_c km/s).
c) pulse velocity within the steel (V_s km/s).

By reference to Fig. 1 it can be shown that for a bar lying parallel to the proposed path, the steel will potentially influence the results when

$$\frac{a}{L} < \frac{1}{2}\sqrt{\frac{V_s - V_c}{V_s + V_c}}$$

in which case, the pulse velocity in the concrete is given by

$$V_c = \frac{2a\,V_s}{\sqrt{4a^2 + (TV_s - L)^2}} \quad \text{km/s}$$

provided that $V_s > V_c$, where T = measured transit time (sec.).

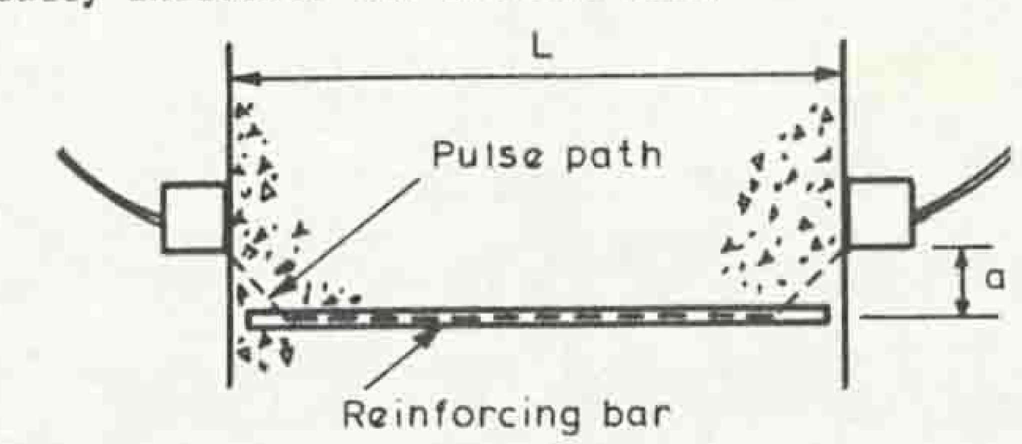

Fig. 1 Influence of longitudinal bar

If the measured pulse velocity in such circumstances is V_m, a correction factor (k) can be developed such that $V_c = kV_m$, with V_s and a/L as variables such that

$$k = \gamma + 2\left(\frac{a}{L}\right)\sqrt{1 - \gamma^2} \quad \text{where } \gamma = \frac{V_c}{V_s}$$

It can be shown that the above expressions will only hold when the offset (a) is large in relation to the end cover (c). If a < 2c (approximately) then the pulse will theoretically pass through the full length of the bar (L_s) and the velocity through concrete is given by:

$$V_c = \frac{2V_s(\sqrt{a^2 + c^2})}{(TV_s - L_s)} \quad \text{km/s}$$

and for the case where a = 0 (i.e. bars directly in line with pulse) the correction factor k may be obtained from

$$k = 1 - \frac{L_s}{L}(1 - \gamma)$$

This same expression will apply to the case of bars transverse to the pulse path, as shown in Fig. 2, in which case the total path length in the steel (L_s) is taken as the sum of the diameters of the individual bars.

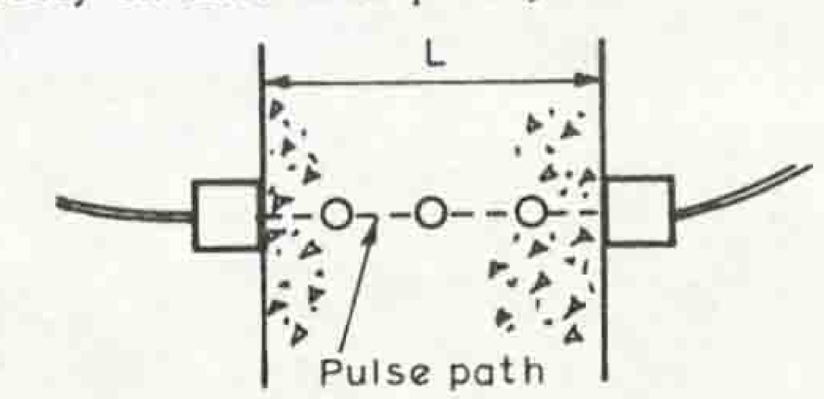

Fig. 2 Transverse bars

The above expressions are based on the assumptions that concrete is a uniform homogenous material, and that the pulse is transferred between the concrete and steel with total efficiency. They also require a value for V_s, the pulse velocity in the steel embedded in the concrete, to be available. Although the value will be influenced by the pulse velocity in the surrounding concrete, no detailed information concerning this is currently available in Standards. B.S. 4408: Pt. 5 [1] provides correction data based on V_s = 5.5 km/s for longitudinal bars.

2.2 Effect of diameter and surrounding concrete

Chung [3] has proposed an effective velocity concept to account for the fact that the pulse velocity in steel varies according to the surrounding medium.

For a bar embedded in concrete, the effective pulse velocity along the bar will be less than for the bar in air and is dependent upon its diameter. Chung has developed the following empirical expression to allow for these combined effects

$$V_s = 5.90 - 10.4(5.9 - V_s)/\Phi \quad \text{where } \Phi \text{ is the bar diameter.}$$

It is claimed that this relationship applies to bars of 10 mm diameter or over, and that the influence of smaller bars can scarcely be detected.

3. TEST PROGRAMME

3.1 Steel Bars in Air

Samples of steel bars of different types and diameters were tested to determine their pulse velocity in air by applying the transducers to the smoothed end faces of the bars. These values are plotted in Fig. 3 for transducers with a frequency of 54 kHz which is the type most commonly used for insitu concrete investigations. Measured values using a frequency of 82 kHz were found to be approximately 2% higher.

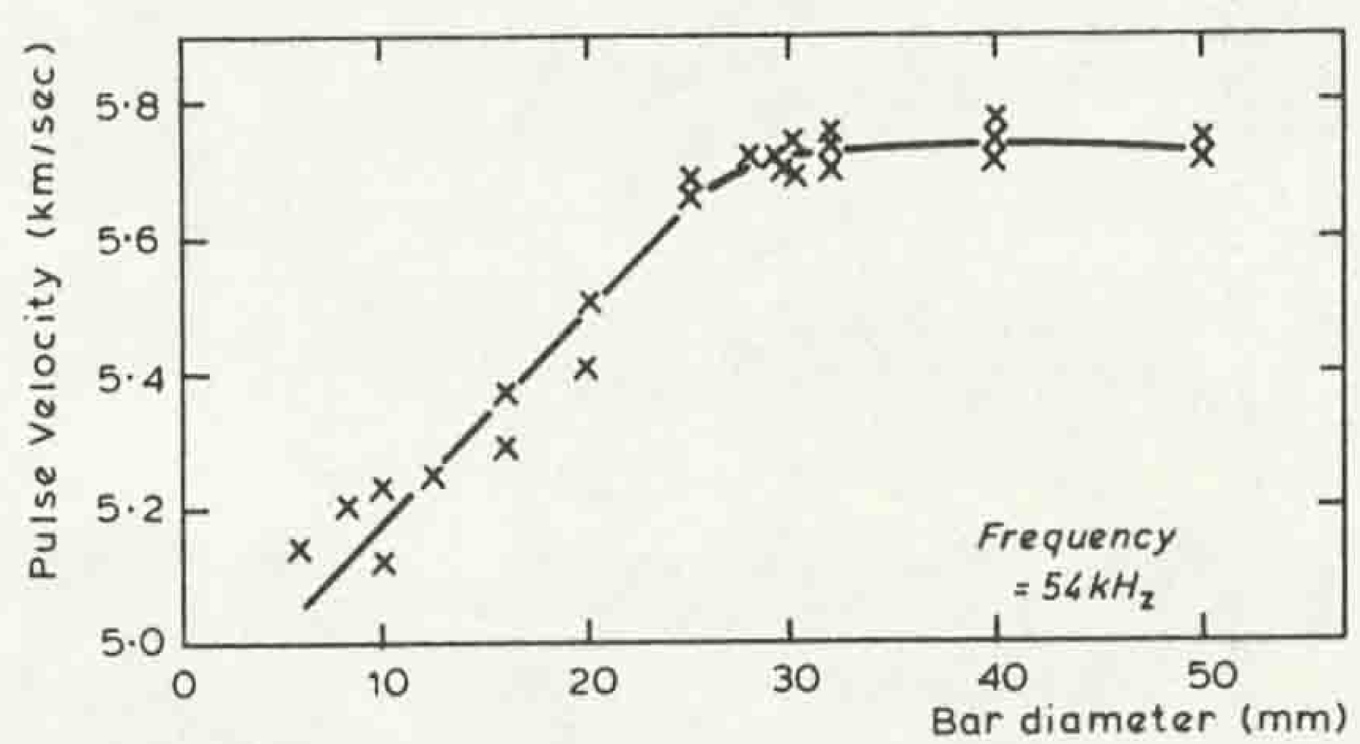

Fig. 3 Results for bars in air

3.2 Steel Bars Embedded in Concrete

A series of eight concrete test specimens with dimensions 490 x 250 x 150 mm were then cast. Each contained embedded reinforcement as shown in Fig. 4, and bar diameters ranging between 6 mm and 50 mm were used. Initially the end cover to the bars was between 85 mm and 130 mm and readings of pulse velocity were taken at increasing ages, commencing at 2 days, to obtain a range of concrete pulse velocities between 3.9 km/s and 4.5 km/s. These measurements were taken directly along the line of each bar, and also transversely across the block in line with each bar. Subsequently the ends of the blocks were sawn off to give end covers in the range 20-25 mm and the longitudinal measurements were then repeated. Round mild steel was generally used since earlier preliminary tests had indicated no significant differences due to steel type, and a pulse velocity of 54 kHz was adopted for these tests.

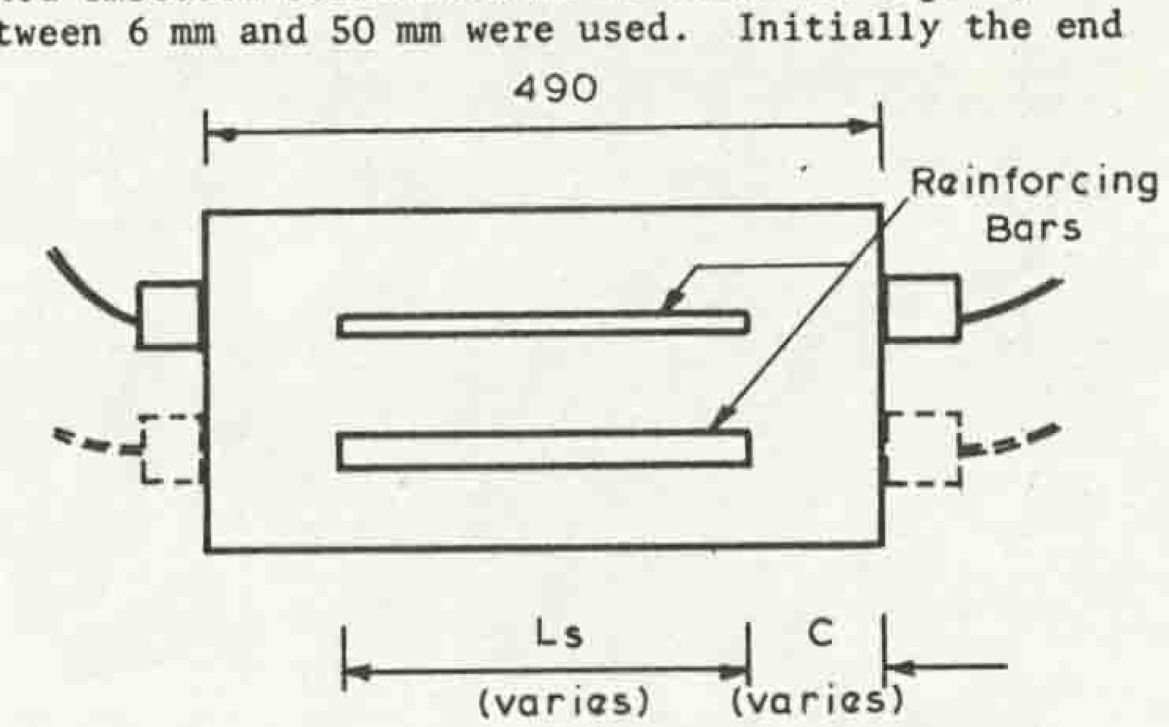

Fig. 4 Test block

3.3 Effects of Cracking and Bond

Following the longitudinal readings with 25-30 mm end cover, a single substantial crack was induced across the full cross-section of each block approximately at the midpoint of its length. The longitudinal readings were then repeated for comparison with the previous values. In addition to two blocks of the series which contained bars liberally coated with grease prior to casting, further specimens in the form of 150 mm cubes were made with comparative greased and ungreased bars cast-in with their ends projecting from the cubes. 10 mm and 32 mm bars were examined in this way.

3.4 Beam containing reinforcement

Following the above tests, an extensive series of readings was taken across the 150 mm width of a 4 m long reinforced concrete beam which had been cast in the laboratory by undergraduate students. Readings were taken at 3 levels, including that of the 12 mm main steel, and were spaced to be in line with, and midway between, the 6 mm links.

4. DISCUSSION OF RESULTS

4.1 Steel Bars in Air

It is clear from Fig. 3 that the pulse velocity along a reinforcing bar in air is reasonably constant for bar diameters of 30 mm and above, with an average value of approximately 5.72 km/s. This reduces with bar diameter below that size in an approximately linear manner.

4.2 Embedded bars parallel to pulse path

The effective velocity in each reinforcing bar has been calculated from the measured values by allowing for the concrete end cover and γ evaluated. Comparisons of the results with established theories are summarised in Fig. 5 which shows the ratio of theoretical/measured velocities in the steel for concrete pulse velocities between 4 km/s and 4.5 km/s. This covers the most commonly occurring range of concrete quality and indicates that the experimental values of V_s generally agree with Chung's predictions for bars of 20 mm diameter or over. For smaller bars, Chung's theory underestimates the steel influence and his contention that bars of 10 mm or less may be ignored is not confirmed.

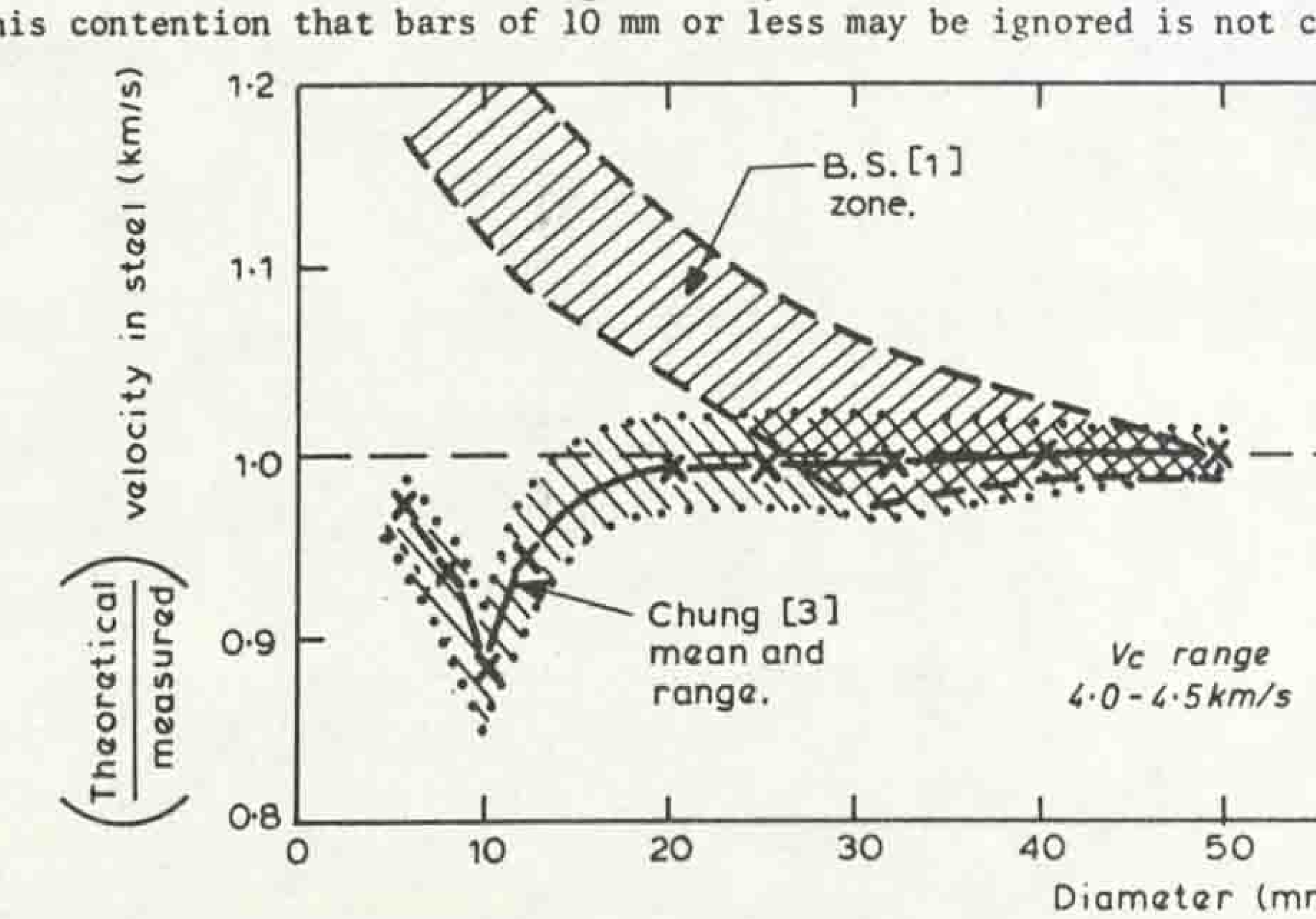

Fig. 5 Results for longitudinal bars

The British Standard values provide a broad band for the range of V_c since V_s is assumed to be constant. The prediction is good for 50 mm bars, and remains reasonable for bars of 30 mm or over in good quality concrete. Below this size it has been shown (Fig. 3) that the bar velocity decreases significantly, and this corresponds well with the features of Fig. 5, which demonstrates the inadequacy of the currently accepted corrections.

The overall range of measured values was approximately 5% for any particular combination but it must be remembered that the generally accepted accuracy of measured pulse velocities on a concrete member is about ± 2%. It was found that the measurements with reduced end cover to the bars yielded higher apparent steel pulse velocities (2% to 3% on average) than those for larger end covers. This may be due to reduced pulse attenuation within the cover concrete, but the influence of differences in contact surface (moulded and sawn) may also have contributed. It should be noted that Chung [3] used 32 mm end cover with moulded contact surfaces.

Measured values of γ are plotted in Fig. 6 for two levels of concrete pulse velocity, and may be used as the basis of a correction factor when used in conjunction with the expression given in section 2.1. The importance of bar diameter is clear, and it will be noted that for small diameter bars the velocity of pulses in the surrounding concrete is of reduced significance. It is also clear that bars of only 6 mm diameter may be detected although their effect is small, and the links of this size could not be identified in the tests on the beam when V_c = 4.4 km/s and $L_s \approx 0.67L$.

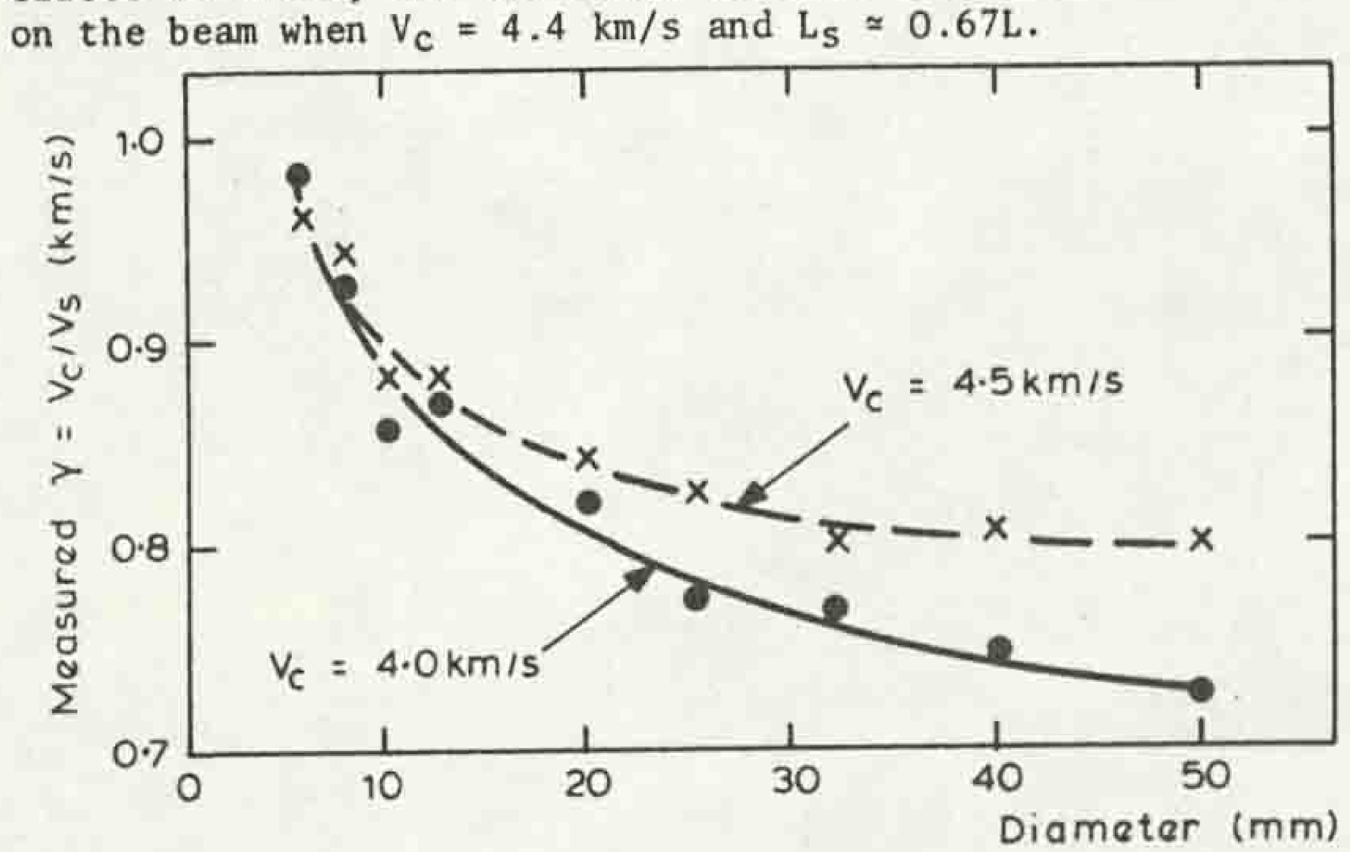

Fig. 6
Effect of V_c and diameter for longitudinal bars

Cracking of the concrete results in a reduction of the measured pulse velocity, which is related to the diameter of the bar concerned. The results are summarised in Fig. 7. This follows the general pattern of Fig. 3 for bars in air, and shows that the effect is greatest for those bars with the largest differential between steel and concrete pulse velocities. Since velocities in bars embedded in concrete are lower than for similar bars in air this result is surprising and suggests that the presence of the crack disrupts the pulse passing through the steel. The increased effect on bars debonded by greasing is perhaps even more surprising.

For the uncracked specimens, the effects of greasing the bars was unexpectedly small, and although there was a general tendency towards higher measured longitudinal pulse velocities in such specimens the difference was only about 1% and not significant in practical terms. A greater effect is likely if the pulse does not enter the bars directly through these end faces.

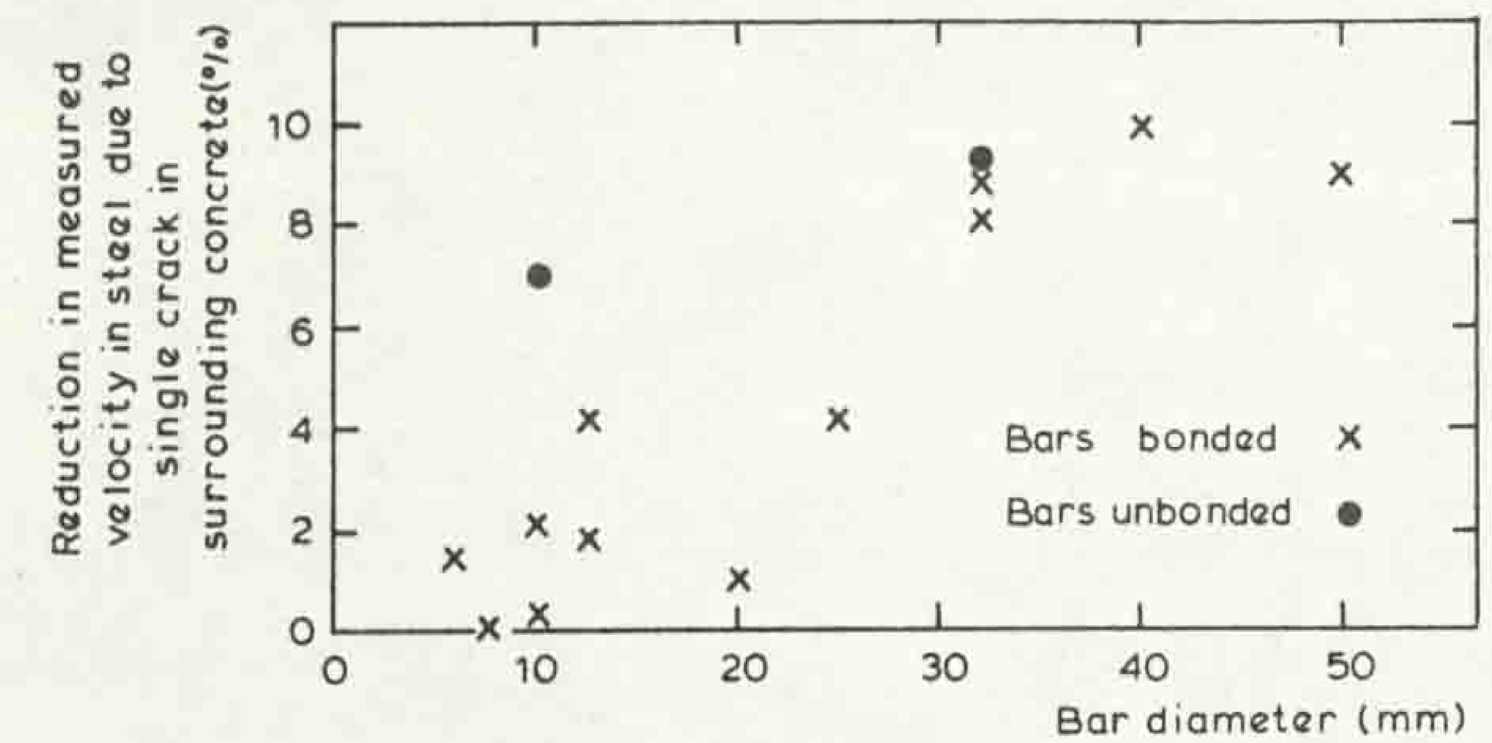

Fig. 7
Effects of cracking

4.3 Transverse Bars

Calculated effective pulse velocities in the steel yield values of low accuracy due to the small path in steel. Consequently, these results have been presented directly in the form of the correction factor k required to convert measured pulse velocity to concrete pulse velocity. For ease of presentation, the values corresponding to L_s/L = 0.2 have been computed from the measured readings and are plotted in Fig. 8 for two values of concrete pulse velocity.

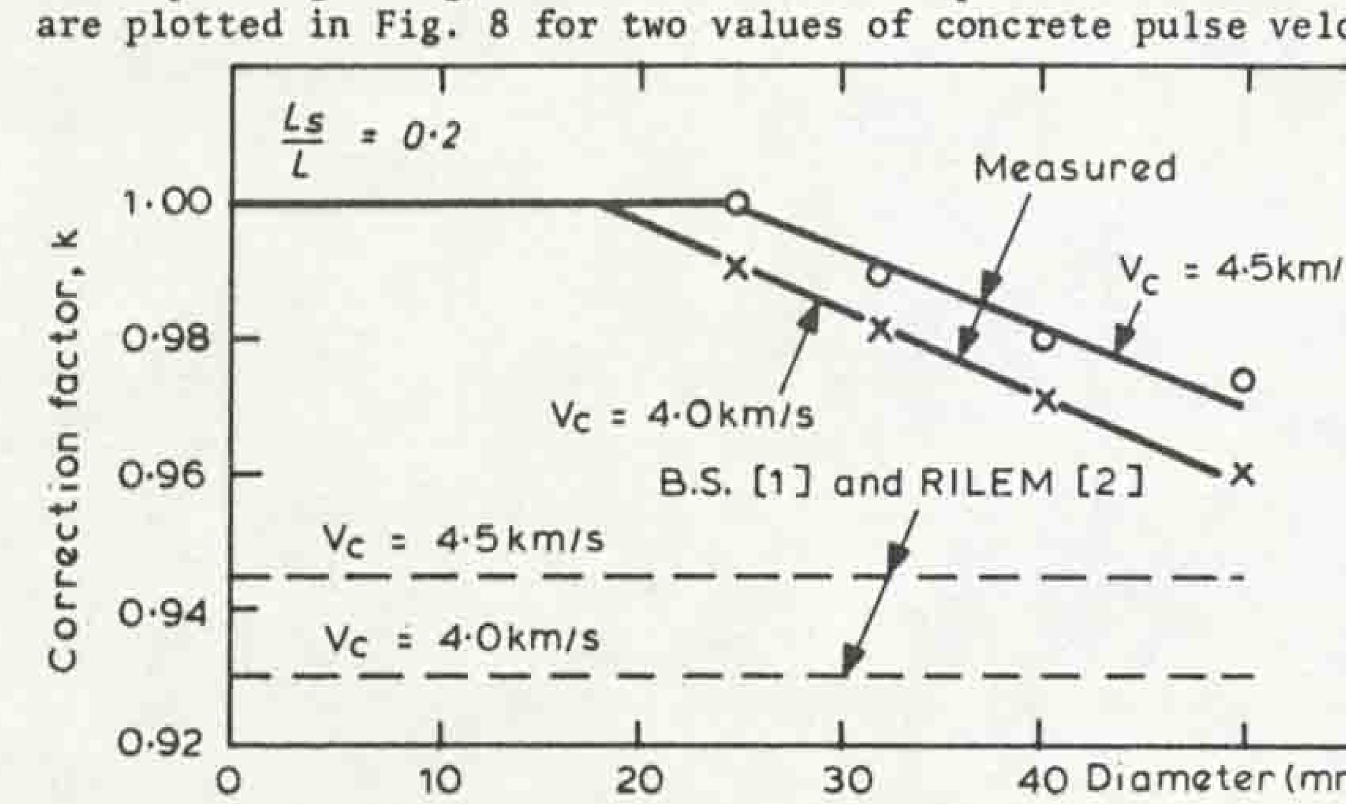

Fig. 8
Correction factors for transverse bars

The inadequacy of existing recommended corrections is again obvious. The effect of reinforcement in this orientation is much smaller than longitudinally, but is nevertheless related to the bar diameter with bars of 20 mm or smaller scarcely detectable. Tests on the beam (section 3.4) showed no influence from the 12 mm main bars. Earlier tests by the Author [4] have indicated that a theoretical estimate of the effect of a transverse bar may be obtained by consideration of the bar as having an equivalent longitudinal length equal to the diameter, but with an effective diameter equal to one half of the true value. Use of the expression proposed by Chung [3] will then yield a value of V_s and hence correction factor k. If this approach is applied to this series of tests, excellent agreement is obtained with the measured values.

The effect of greasing on transverse readings was found to be dramatic, with the result that the influence of the bar disappeared completely. The pulse in this situation is unable to effectively enter the steel, which becomes equivalent to a void of insufficient size to be detected.

5. CONCLUSIONS

The transfer of ultrasonic pulses between concrete and embedded steel is complex, and it is virtually impossible to make allowances for such steel with precision. Nevertheless, the currently accepted recommendations for making such allowances are seriously in error except for large diameter bars. The effect of a 10 mm bar lying along the line between the transducers may, for example, be over-estimated by more than 15%. An error of this magnitude totally negates the value of pulse velocity readings as an indicator of concrete properties since most practical concretes will have values lying within an over-all range of 20%.

If the bar does not lie directly in line with the transducers, the pulse path is less clearly defined and transfer to the concrete is likely to be less efficient than through the bar ends. Cracking, which is quite likely in regions of main reinforcement, will further reduce the reliability of results where longitudinal steel is present.

Small diameter bars, such as those commonly used for links and binders may have a significant influence, and where longitudinal steel cannot be avoided it may be allowed for by the use of Fig. 6 to obtain a suitable correction factor. An estimate of concrete pulse velocity obtained in this way may be expected to be accurate within ± 3% provided that cracking is not present and precise details of the steel are known.

Transverse steel has been found to have a much smaller effect than predicted by existing methods, and 20 mm diameter bars, or smaller, may be ignored. Given an accepted error of ± 2% on measured pulse velocities, it is possible that the influence of 25 mm bars may also be regarded as insignificant for practical purposes. Reliable allowances may be made on the basis of Fig. 8 or the numerical procedure described in section 4.3 provided that the bars are well bonded to the concrete. This will represent the most common practical usage of reinforcement corrections.

These results were obtained under laboratory conditions using portable commercially available equipment. It is likely however that in practice the effects of reinforcement will be less readily detected due to a combination of reduced accuracy of on-site measurement and the intrinsic variations of concrete properties within a structural member [4]. Where reinforcement details are known the use of the corrections proposed here will nevertheless provide an indication of the true properties of the concrete in the test zone with much greater reliability than other available methods. If the concrete pulse velocity is known, it may also be possible in some circumstances to obtain an estimate of the quantity of embedded steel as well as an indication of its presence.

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Paper 9

"The Influence of Reinforcement on Ultrasonic Pulse Velocity Testing"

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The Influence of Reinforcement on Ultrasonic Pulse Velocity Testing

By J. H. Bungey

Synopsis: Embedded reinforcement may have a significant effect on ultrasonic pulse velocity measurements taken through structural concrete members. Reliable corrections are essential if test locations cannot avoid the influence of the steel. Extensive laboratory experimental work demonstrates major shortcomings in all currently accepted allowance procedures and confirms that bar diameter is an essential variable to be incorporated. The effect of bars passing across the pulse path is shown to be less than for bars of similar size running along the path. A correction procedure is proposed which can meet many practical combinations of bar size, bar orientation and concrete properties with significantly greater accuracy than possible by established methods.

KEYWORDS

CONCRETE   TESTING   INSITU   ULTRASONIC   REINFORCEMENT

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INTRODUCTION

It is well established that reinforcement which is located along, or close to, the line of ultrasonic pulse velocity measurements will have a significant influence upon the measured values. Pulses will generally travel more quickly within steel that concrete and may travel up to about 1.9 times faster in a reinforcing bar than in the surrounding concrete. The most reliable practical approach is that reinforcement should be avoided whenever possible in selecting test locations. There will, however, often be circumstances in which this is impossible and it is then necessary to make a correction to the measured value to give an estimate of the velocity of the pulses in the plain concrete. Such corrections are not easy to establish, and the influence of the steel may dominate over the properties of the concrete when taking measurements.

The correction factors that have been accepted in Europe (1) are those currently recommended by British Standards (2) and RILEM (3). These are essentially similar and involve only two basic parameters which are the pulse velocity within the concrete and the relative path lengths within the steel and concrete. Chung (4, 5) has more recently demonstrated the importance of bar diameter, and proposed a procedure which takes this into account for bars running in the same direction as the pulse. The writer (6) has subsequently compared these values with his own experimental results and demonstrated the severe shortcomings of all current recommendations. The experimental work has now been extended to provide data forming the basis of a comprehensive correction procedure which is presented in this paper.

NOTATION

a	perpendicular distance from edge of steel bar to nearest edge of transducer
c	cover thickness
k	pulse velocity correction factor
L	shortest distance between transducers
L_s	maximum path length within steel
T	measured transit time
V_c	pulse velocity in plain concrete
V_m	apparent pulse velocity in concrete where steel is present

V_s effective pulse velocity in reinforcing steel

x distance from concrete surface to pulse entry point on bar

γ pulse velocity ratio = V_c/V_s

ϕ bar diameter

BASIC THEORY

The basic geometric aspects of the problem are well established, and are briefly summarized here. The timing circuits of the ultrasonic pulse measurement apparatus will normally respond to the leading edge of the first longitudinal or compression wave. Thus, the measured transit time will relate to that component of the pulse which has taken the fastest route between the transmitting and receiving transducers. This may pass through any steel which lies sufficiently close to the direct path between transducers. The geometric configuration necessary for this to occur depends upon the relative path lengths through concrete and steel, and upon the relative pulse velocities through the two materials.

For the situation shown in Figure 1, it is assumed that the concrete exhibits a uniform pulse velocity V_c and that the pulse velocity in the steel V_s is higher. Established theories also assume that the pulse is transferred at the outer edges of the contact area between transducers and concrete, and will travel along the nearest surface of the reinforcing bar. It can then be demonstrated (4) that the first component to reach the receiver will theoretically travel through the steel if the ratio of the offset distance to the length of the direct path through the concrete is such that

$$\frac{a}{L} < \frac{1}{2}\sqrt{\frac{V_s - V_c}{V_s + V_c}} \qquad (1)$$

In such cases the pulse velocity in the concrete can be calculated from the measured transit time using

$$V_c = \frac{2aV_s}{\sqrt{4a^2 + (TV_s - L)^2}} \qquad (2)$$

If the offset is small, it can be seen that the quickest path will include the whole length of the bar and the pulse velocity in the concrete is then given by

$$V_c = \frac{2V_s\sqrt{a^2 + c^2}}{(TV_s - L_s)} \qquad (3)$$

The transition between these two expressions may be obtained by evaluation of the distance x from the concrete surface to the point at which the pulse would theoretically enter the steel bar for the minimum transit time. This may be conveniently expressed as

$$x = \frac{\gamma}{\sqrt{1 + \gamma^2}}\, a \qquad (4)$$

where $\gamma = V_c/V_s$ (5)

Limiting values of the ratio a/c can thus be obtained such that $x \geq c$, as illustrated in Figure 2. It follows therefore that for most practical circumstances in which steel corrections are being made, Equation (2) will apply for ratios $a/c > 2.0$ and Equation (3) for $a/c < 1.4$. If the ratio lies between these values reference should be made to Figure 2 to determine the appropriate expression according to γ, the ratio of pulse velocities in the two materials.

Where it is established that reinforcing steel is likely to influence the results, a correction factor k can be developed which relates the true pulse velocity within the concrete to the measured apparent velocity V_m such that

$$V_c = k\, V_m \qquad (6)$$

V_s and a/L are variables in the general case when Equation (2) applies and it can be shown that (4):

$$k = \gamma + 2\left(\frac{a}{L}\right)\sqrt{1 - \gamma^2} \qquad (7)$$

This is presented graphically in Figure 3, which also indicates the zone of influence of the steel related to γ.

For small a/c ratios, when Equation (3) applies

$$k = \gamma + 2\left(\frac{\sqrt{a^2 + c^2} - \gamma c}{L}\right) \qquad (8)$$

and in the extreme when a = 0 this simplifies to

$$k = 1 - \frac{L_s}{L}(1 - \gamma) \qquad (9)$$

Equation (9) can also apply to bars which are transverse to the pulse path as shown in Figure 4, when the path length in the steel L_s is taken as the sum of the diameters of the individual bars.

The use of correction factors in this form requires knowledge of the pulse velocity in the embedded steel. For a reinforcing bar in air it has been shown (6) that the velocity is likely to be about 5.75 km/s for bars of 30mm diameter or greater, reducing linearly to 5.05 km/s for 6mm bars. These values are reduced when the bars are embedded in concrete, depending upon the pulse velocity within the concrete. Reliable determination of this relationship can only be achieved experimentally, although Chung (4) has proposed an empirically based general relationship for longitudinal bars such that

$$V_s = 5.90 - 10.4\,(5.90 - V_c) / \Phi \qquad (10)$$

EXPERIMENTAL PROGRAMME

Three types of laboratory test specimen were used. 500 x 100 x 100 mm prisms and 490 x 250 x 150 mm beams with reinforcing bars totally embedded, and 225 x 225 x 225 mm cubes with bars projecting on opposite faces. Four series of tests were performed as summarised in Table 1. Bar diameters ranging between 6mm and 50mm, were combined with a variety of concrete mixes proportioned to give a wide spread of concrete pulse velocities. Values between 3.0 km/s and 5.0 km/s were achieved, with lightweight aggregates yielding the values at the lower end of this range. Other mixes used a hard gravel aggregate. A variety of steel types including plain round mild steel and ribbed high yield point bars were compared with identical greased bars. End cover to the bars was varied between 110mm - 20mm in the Series A specimens by diamond saw cutting, and the blocks were later cracked by application of a concentrated transverse load. The Series D tests, which were principally aimed at providing transverse results, also compared square and round bars of comparable dimensions in varying quantities (L_s/L from 0.09 to 0.89). Tests were conducted at ages between 1 day and 1 year, with varying moisture conditions, to produce a large quantity of overlapping data from which the average behaviour and variability of particular combinations of bar size and concrete pulse velocity can be predicted.

Values of pulse velocity in the plain concrete were obtained from measurements outside the zone of steel influence on all specimens, and also upon companion plain concrete prisms for Series B and C tests. All tests were performed using 50mm diameter piezoelectric transducers of 54 kHz nominal frequency with a light grease couplant, in conjunction with digital reading equipment of British make.

EXPERIMENTAL RESULTS

The majority of measurements were taken with the reinforcing bar lying directly on the path between transducers, but some offset readings were also made for the Series A and D tests. In each case the value of pulse velocity within the steel was evaluated by deducting from the measured transit time the time taken by the pulse to pass through the appropriate path length of concrete. The velocity in the concrete was then divided by the calculated steel velocity to obtain the value of γ relating to the particular combination of bar diameter, orientation, and concrete pulse velocity.

Longitudinal Bars

The results of all tests with the bar running directly along the path between transducers are shown graphically for each bar size in Figures 5 and 6. It can be seen that in every case, despite experimental scatter, the results from each of the Series A, B and C fit together well to give a linear variation of the ratio γ with concrete pulse velocity for a particular size. If $V_s < V_c$ the presence of the bar cannot be detected, hence there is a practical upper limit of $\gamma = 1.0$.

The combined effects of V_c and bar diameter are clearly illustrated in Figure 7 in which curves for specific values of V_c have been developed from the data of Figures 5 and 6. The influence of the reinforcement is greatest when γ is smallest, as for large diameter bars in concrete having a low pulse velocity. The relationships are clearly defined for large diameter bars, but for diameters below 10mm the value of γ changes rapidly with V_c and bar diameter. Bars below 6mm diameter cannot normally be detected.

Transverse Bars

The results of all tests with bars running directly across the path between transducers are summarised in Figure 8. Similar patterns are observed, but with a reduced magnitude of steel influence and with greater scatter. This increased scatter is partially due to the inaccuracies of calculating V_s for small values of the ratios L_s/L, and it was noted that the lowest values of γ were generally associated with the higher L_s/L ratios. Bars below 20mm diameter cannot normally be detected, and for values of V_c of above 4.0 km/s 20mm bars have no practical influence. Figure 9 illustrates the combined effect of bar diameter and concrete pulse velocity in a similar manner to Figure 7 for the longitudinal bars. The reduced effective velocity in the steel may be due to the fact that only a proportion of the path length in the steel will relate to the full bar diameter, combined with the short path length across each bar.

Other Features of Test Results

Measurements on Series D specimens to compare square and round bars of similar dimensions surprisingly indicated no major difference in effect either transversely or longitudinally. Comparisons in Series A and D between round mild steel and ribbed high yield point steel similarly indicated no detectable difference. Where debonded bars were compared with identical bonded bars only a small effect was noticed longitudinally, but the influence of debonded transverse bars disappeared completely since the bar effectively becomes a void which is too small to detect. The tests upon Series A specimens to examine the effect of end cover thickness showed no significant variations due to this. However, subsequent tests after a single transverse crack had been formed across each block at the midpoint of its length showed a significant reduction in the measured velocity along the reinforcement. This reduction increased with bar diameter and was of the order of 10% for large diameter bars. Not surprisingly, longitudinal values of V_s were found to be higher when the transducers were in direct contact with the steel as in Series D tests.

Results for readings taken offset from the line of reinforcement were limited in scope and are not presented in detail. These were, however, checked in each case with Figure 3 for the appropriate a/L ratio and value of γ obtained from Figure 5, 6 or 8. Reasonable agreement was found for the zone of influence of bars parallel to the pulse path, but transverse bars showed a much reduced effect since there is not a continuous path in the steel. Theoretical evaluation of this case must incorporate the ratio of the path lengths in concrete and steel as well as offset distance and γ.

CORRECTION FACTORS

Values of γ for specific combinations of bar diameter, orientation and concrete pulse velocity may be obtained from Figures 7 and 9 if it is assumed that the bars are bonded normally within uncracked concrete. A correction factor relating V_c to the measured 'apparent' pulse velocity may then be evaluated from Equation (9) if the bars lie on the direct path between transducers. Despite the greater scatter of results for transverse bars shown in Figure 8, the predicted value of V_c has been found to have 95% confidence limits of less than ±3% irrespective of orientation. The significance of inaccuracies in estimates of γ decreases with the ratio L_s/L which is likely to be smaller for transverse than for longitudinal bars, and the application of this approach to transverse bars is more reliable than the approximate method previously suggested by the writer (6) in which the bars are treated as equivalent longitudinal bars of one half their true diameter.

If longitudinal bars are offset and lie within the zone of influence defined by Figure 3, the correction factor may be obtained directly from that figure provided that the ratio of a/c is above the limit indicated by Figure 2. If this limit is not met, the correction factor given by Equation (8) must be used. In many practical situations there may be uncertainties about the precise location and end cover thickness to reinforcing bars, which will reduce the confidence that may be placed upon correction factors obtained in this way. The increased complexity of the case of offset transverse bars suggests that this situation should be avoided. The zone of influence will however be less than suggested by Figure 3 in most practical circumstances.

The number of variables involved means that it is not possible to directly evaluate a correction factor in the common practical situation where the steel details are known but V_c is unknown. An estimate of V_c must first be made, and then compared with the value obtained from the above calculation procedure using this estimate. The true value of V_c will correspond to the case where the two values are the same, and this may be achieved iteratively. The most efficient approach will be to evaluate and plot a few trial cases selected to encompass the anticipated result.

Ultrasonic pulse velocities are frequently used on site for comparative surveys, and in these circumstances corrected values in the region of reinforcement will be compared with known values of V_c for other regions. Initial corrections may be easily achieved on the basis of the known values, and further examination will only be necessary if unusual discrepancies are found.

Where two way reinforcement is encountered this may invalidate correction procedures, but bars of less than 20mm diameter transverse to the pulse path are unlikely to have any effect upon measured values. It is to be expected that the longitudinal bars will generally provide the dominant effect, but no firm experimental evidence is available to support this view. Attempted corrections in such situations must therefore be treated with great caution. No test data are available for the case of bars running diagonally across the pulse path, and corrections should not be attempted.

COMPARISON WITH OTHER METHODS

Comparisons are made in Figure 10 of corrections for two typical cases of a longitudinal bar in line with the transducers. The BSI (2) and RILEM (3) recommendations are similar and based upon a constant value of V_s of approximately 5.5 km/s, whilst Chung's results involve varying V_s based on Equation (10) but with the steel influence disappearing for 10mm bars. There is reasonable agreement between approaches for large bars but discrepancies increase as the bar size reduces. Discrepancies are also greatest for concrete with a low pulse velocity. The inadequacies of the BSI/RILEM values are obvious, leading to errors of more than 30% in estimated pulse velocity when longitudinal bars are present. To put this in perspective it should be noted that a variation of 2% in pulse velocity is often regarded as significant in terms of other properties of concrete. Reasonable agreement is obtained between the Author's and Chung's results for high pulse velocity concrete except for 10mm bars. However, discrepancies are greater when the concrete pulse velocity is low, and may exceed 20% for 10mm bars.

Comparisons of corrections for typical transverse bars are made in Figure 11. Again, the discrepancies may be considerable, although less dramatic than for longitudinal bars. Chung's results do not extend to transverse bars.

The use of current BSI/RILEM factors, which are stated to represent the maximum likely influence of steel, will thus lead to a substantial under-estimate of true pulse velocity in the concrete except for large diameter bars. Chung's approach may conversely lead to an over-estimate for longitudinal bars which are less than 20mm diameter. Both of these situations may lead to serious misinterpretation of insitu test results.

CONCLUSIONS AND RECOMMENDATIONS

Allowances for the effects of embedded reinforcing bars must take the bar diameter into account. Correction factors which do not do this may under-estimate concrete pulse velocities by up to 30% for small diameter bars. Other correction methods which take diameter into account will tend to under-estimate the effect of small diameter bars running parallel to, or along, the test path, thus leading to an over-estimate of concrete pulse velocity. Bars of this type as small as 6mm diameter may have a significant effect, particularly in concrete with a low pulse velocity. Bars running perpendicular to the pulse path will have a much smaller effect, and for most practical purposes this will be negligible for diameters below 20mm irrespective of how many bars are present. The influence of larger transverse bars of up to 50mm diameter will depend upon their size and quantity, and concrete properties, but if they are not bonded to the surrounding concrete they will not be detectable.

Reinforcing steel should ideally be avoided by ensuring that pulse paths are located outside of its zone of influence. This zone can be predicted theoretically, and test results suggest that the practical extent for longitudinal bars is approximately as predicted. The zone of influence of transverse bars is significantly less than for comparable longitudinal bars. If the influence of steel cannot be avoided, uncertainties and loss of accuracy will inevitably be introduced when assessing the pulse velocity of concrete.

Correction factors developed from the results presented may be applied to many practical situations of reinforcing bars within the range of 6mm-50mm diameter embedded in concrete with a pulse velocity between 3.0 and 5.0 km/s. These can be applied to bonded bars aligned along or across the pulse path, and may be considered to provide an accuracy of estimated concrete pulse velocity of ±3% under idealized laboratory conditions provided that the concrete is uncracked. Under field conditions uncertainties about precise bar location, the condition of bond, and the presence of cracking may reduce the confidence that may be placed on corrections. Where complex bar arrangements occur in the test zone the results must be treated with particular caution, especially if heavy two way steel is present.

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Table 1

Summary Of Test Specimens

Series	Specimen Dimensions (mm)	No. of Specimens	Agg. Type and Size (mm)	Range of V_c (km/s)	Range of bar sizes (mm)
A	490 x 150 x 250	8	Gravel 10, 20	3.87 - 4.65	6-50
B	500 x 100 x 100	8	Gravel 20	4.00 - 4.90	10-32
C	500 x 100 x 100	8	'Lytag' 8	3.00 - 3.50	6-50
D	225 x 225 x 225	30	Gravel 10	3.60 - 4.55	20-50

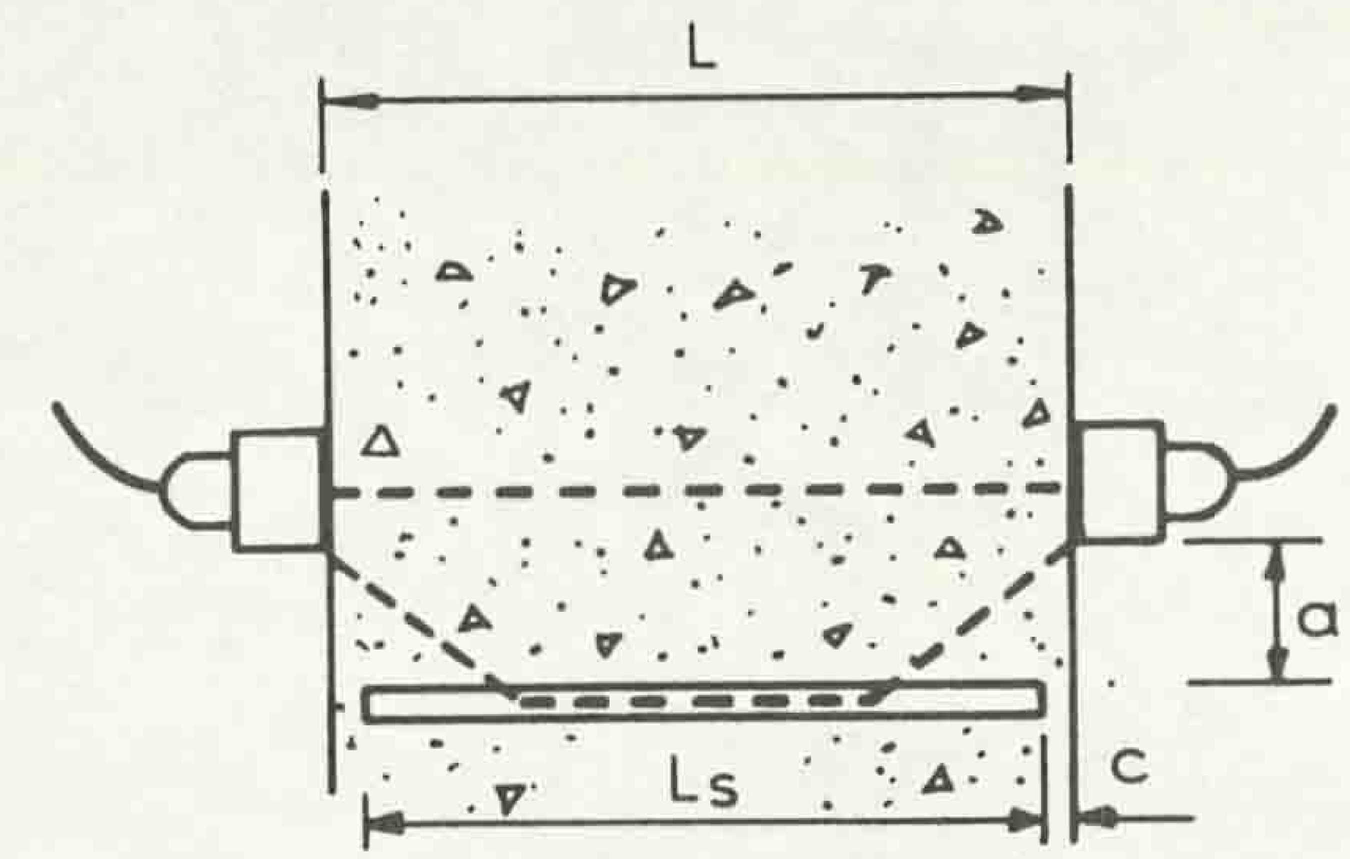

Figure 1. Bar Parallel to Direct Pulse Path.

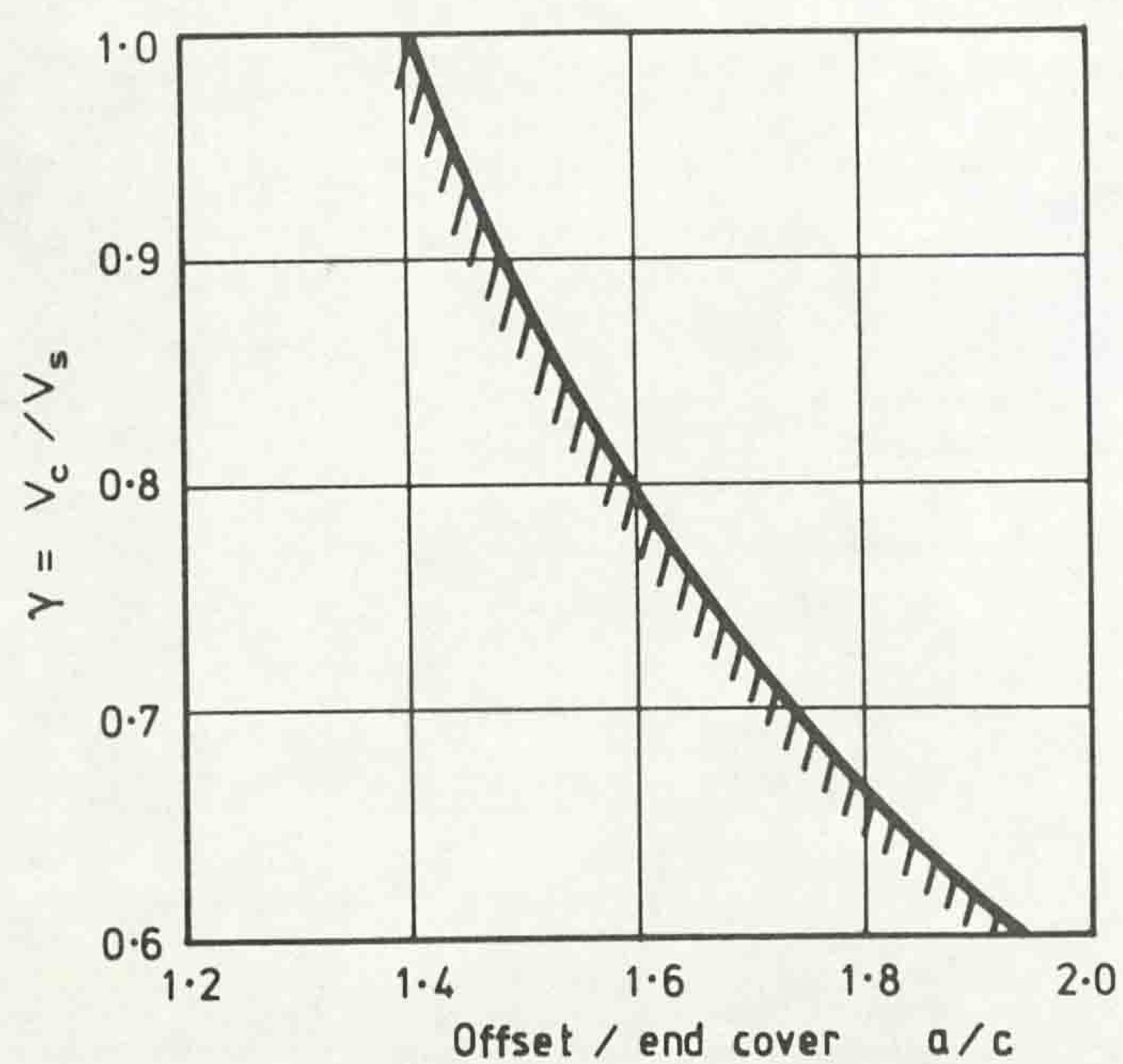

Figure 2. Limiting Ratio of Offset to Cover.
(developed from Equation 4)

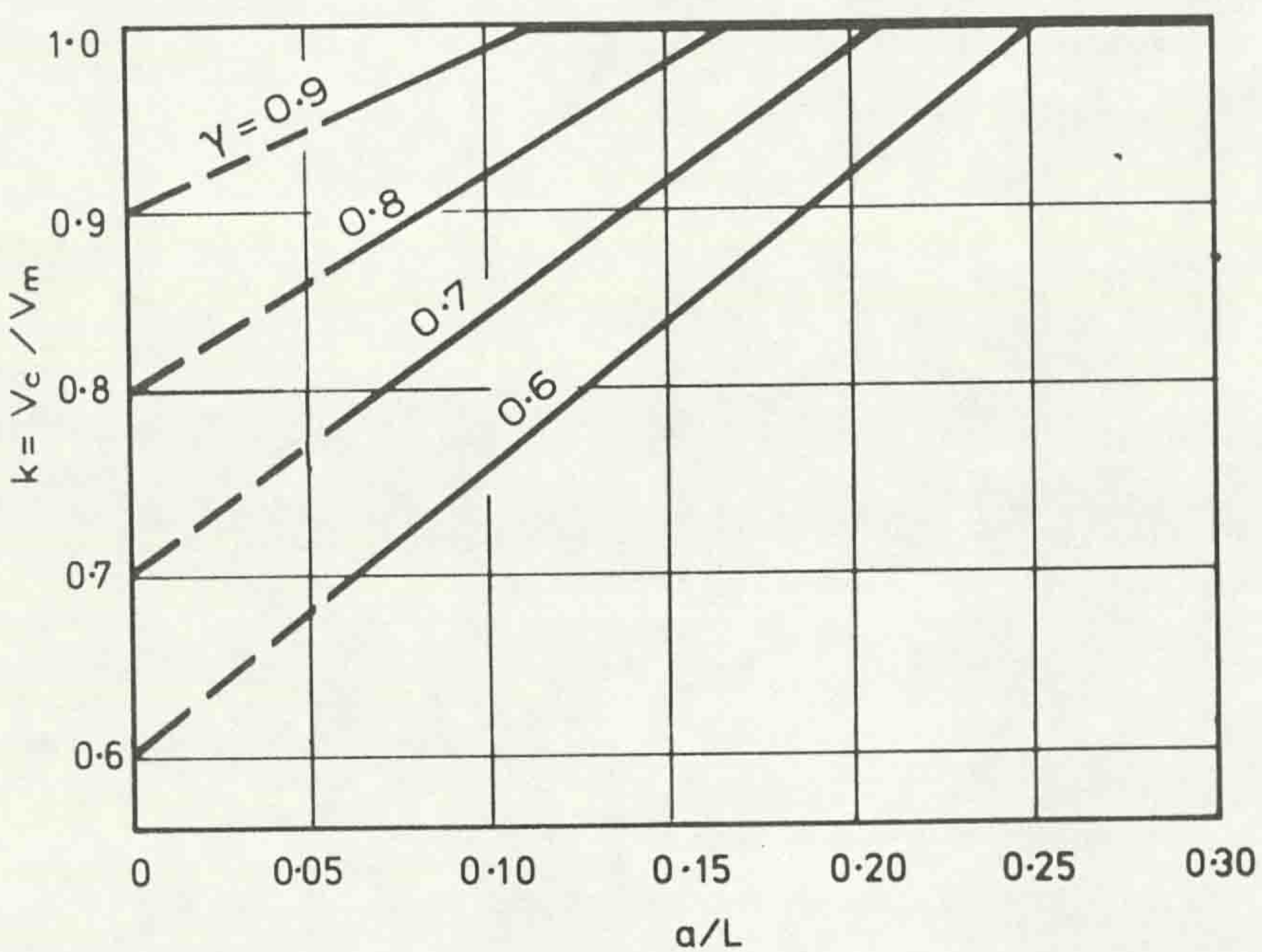

Figure 3. General Relationship between Correction Factor and Other Variables. (Equation 7)

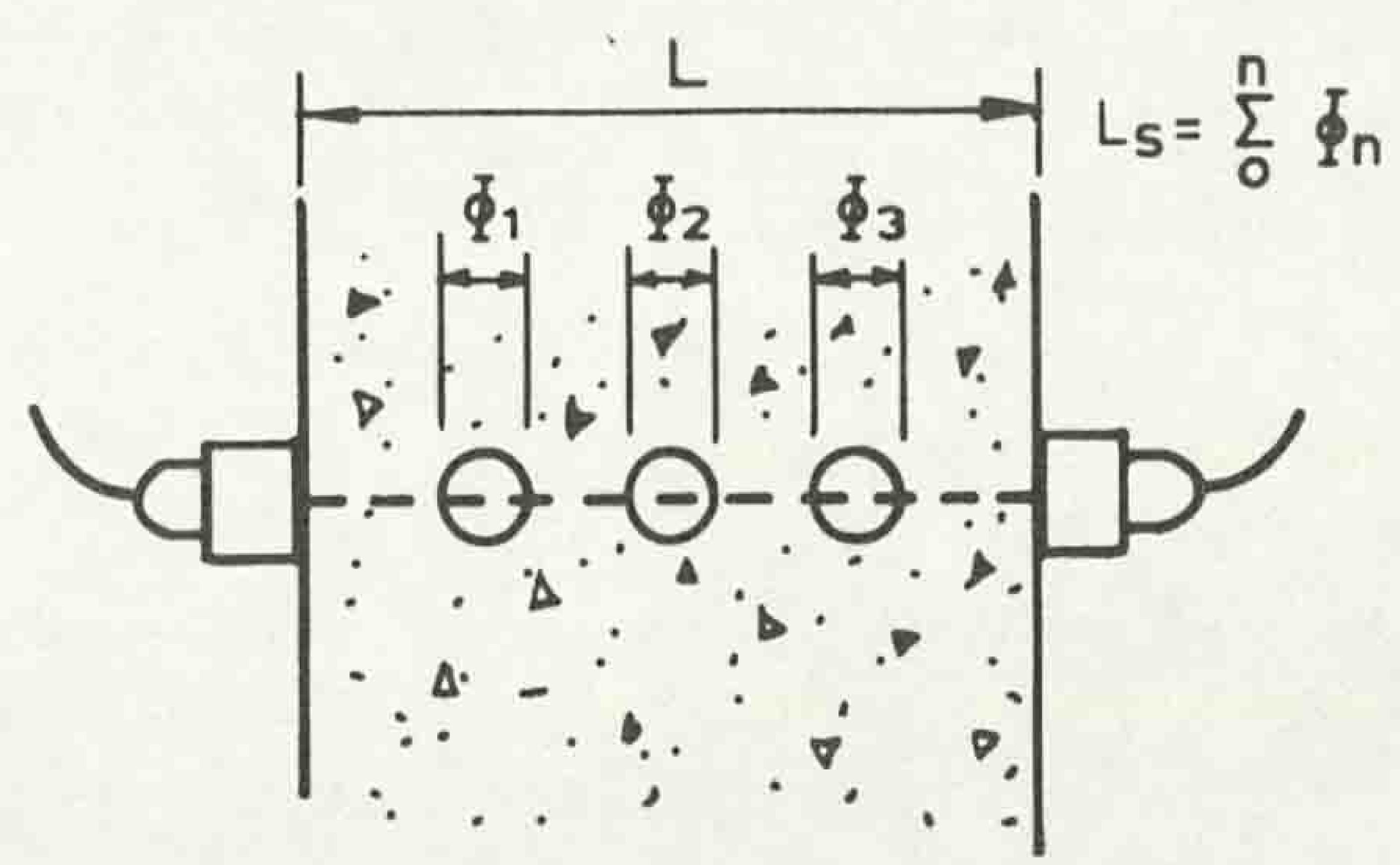

Figure 4. Transverse Bars on Pulse Path.

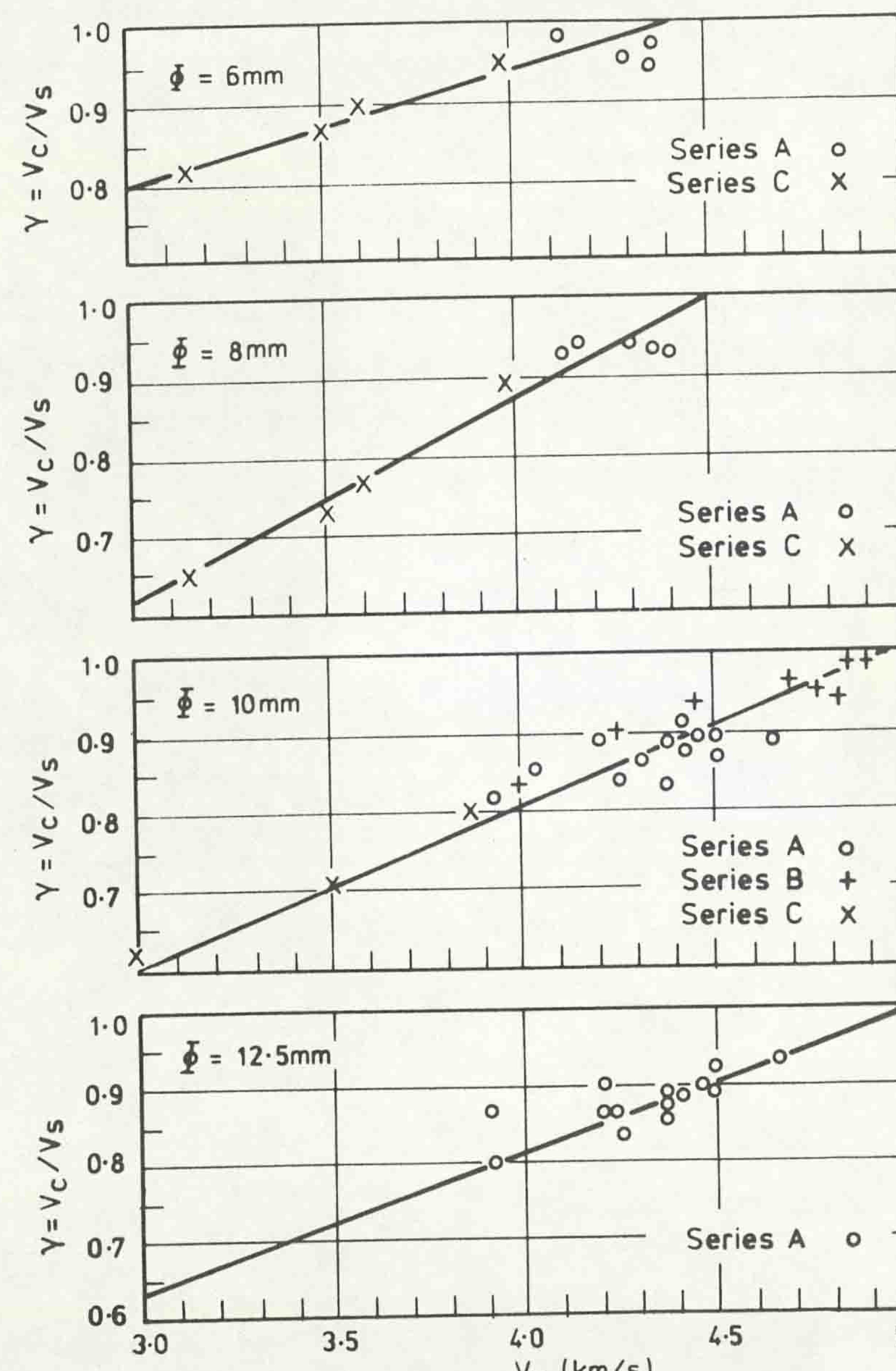

Figure 5. Longitudinal Bar Results.

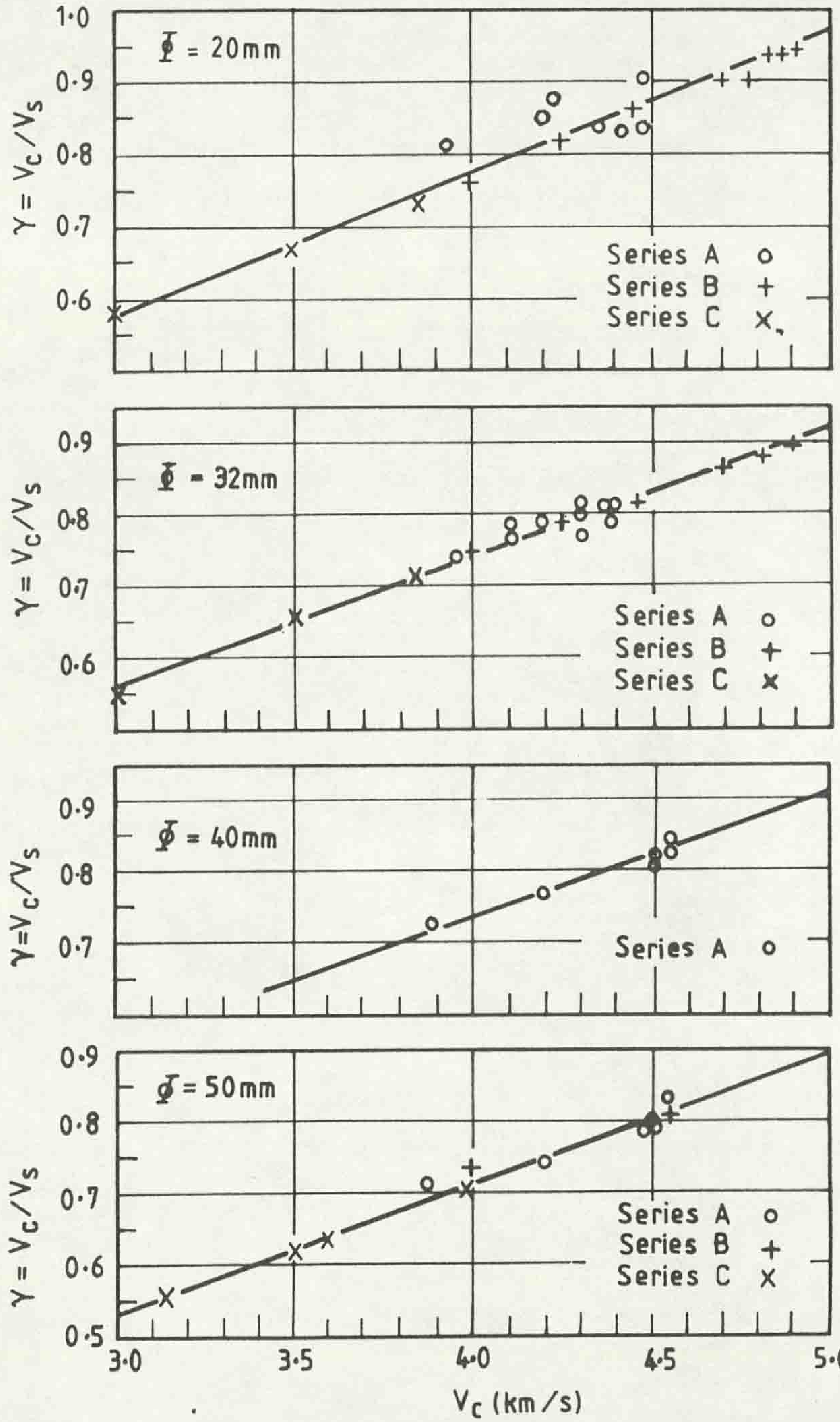

Figure 6 Longitudinal Bar Results.

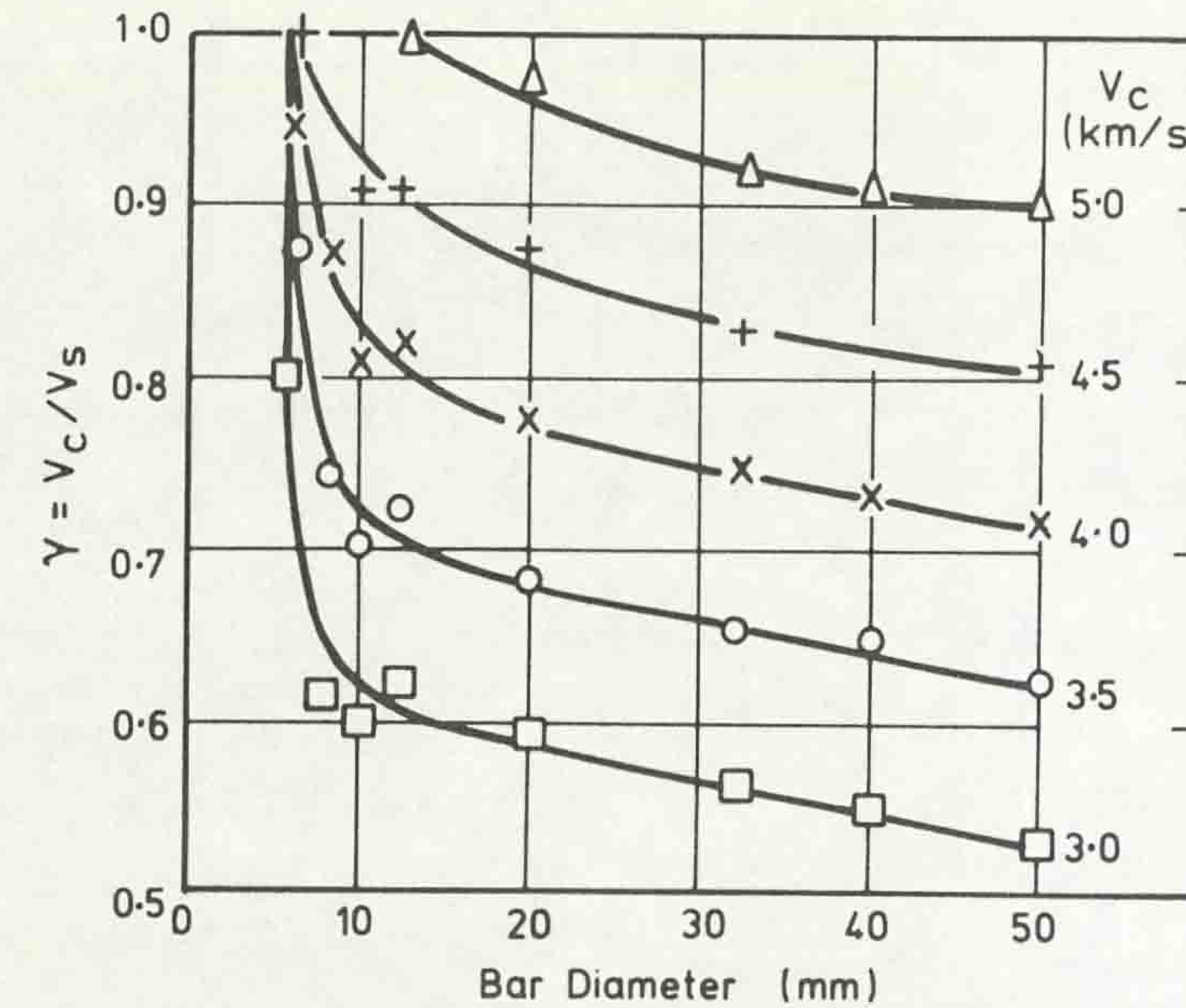

Figure 7. Relationship between γ and diameter for Longitudinal Bars.

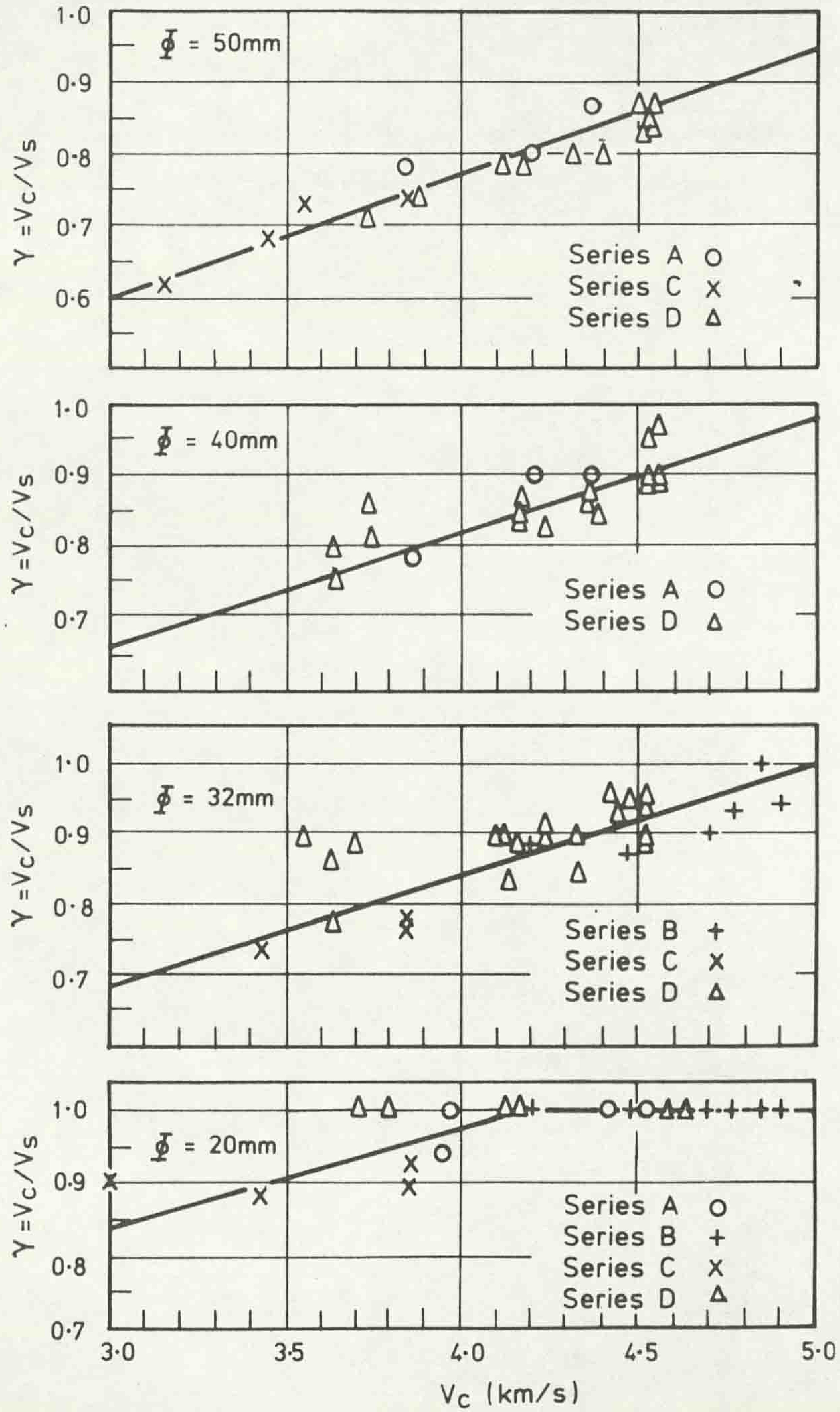

Figure 8. Transverse Bar Results.

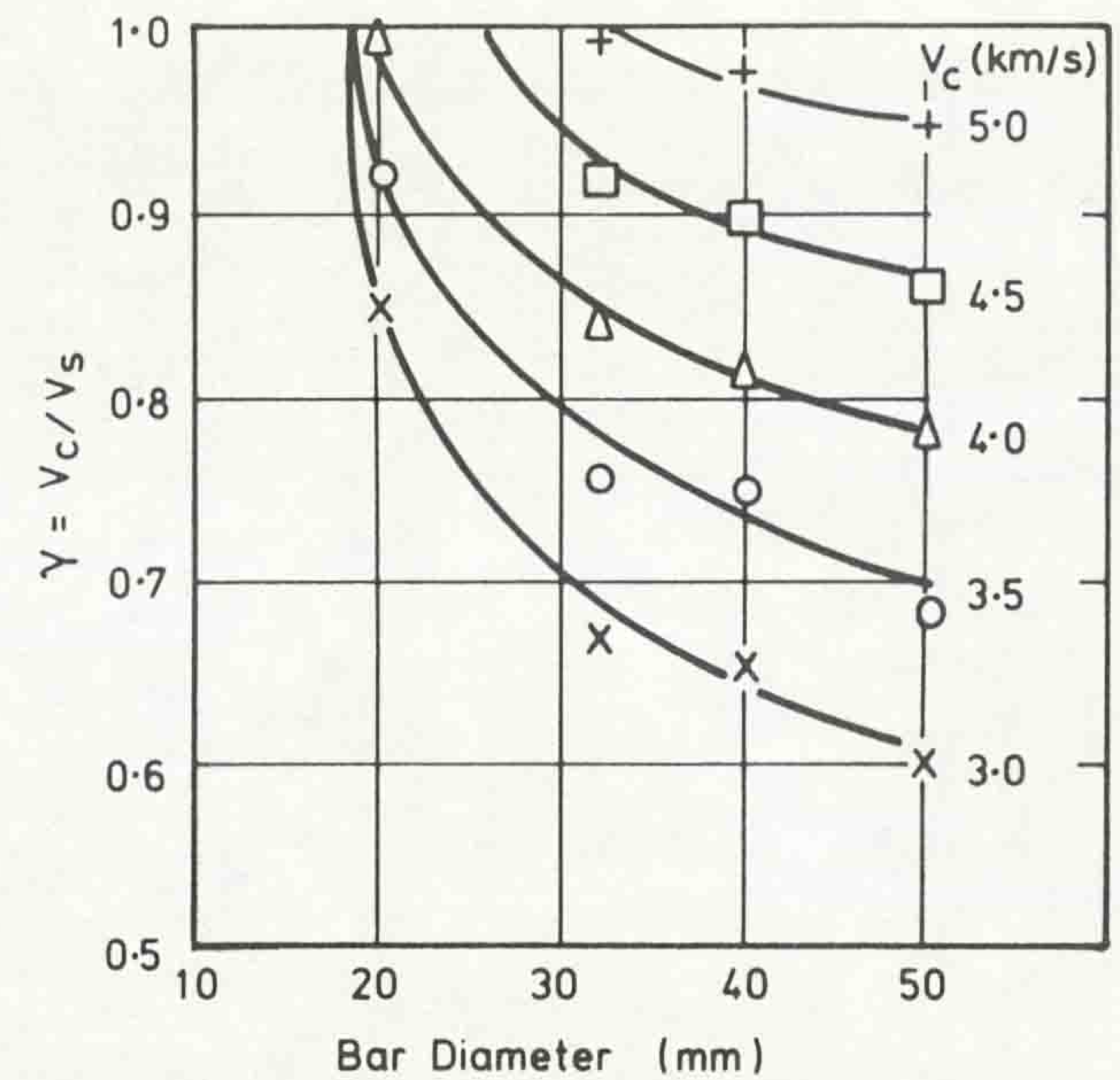

Figure 9. Relationship between Y and diameter for Transverse Bars

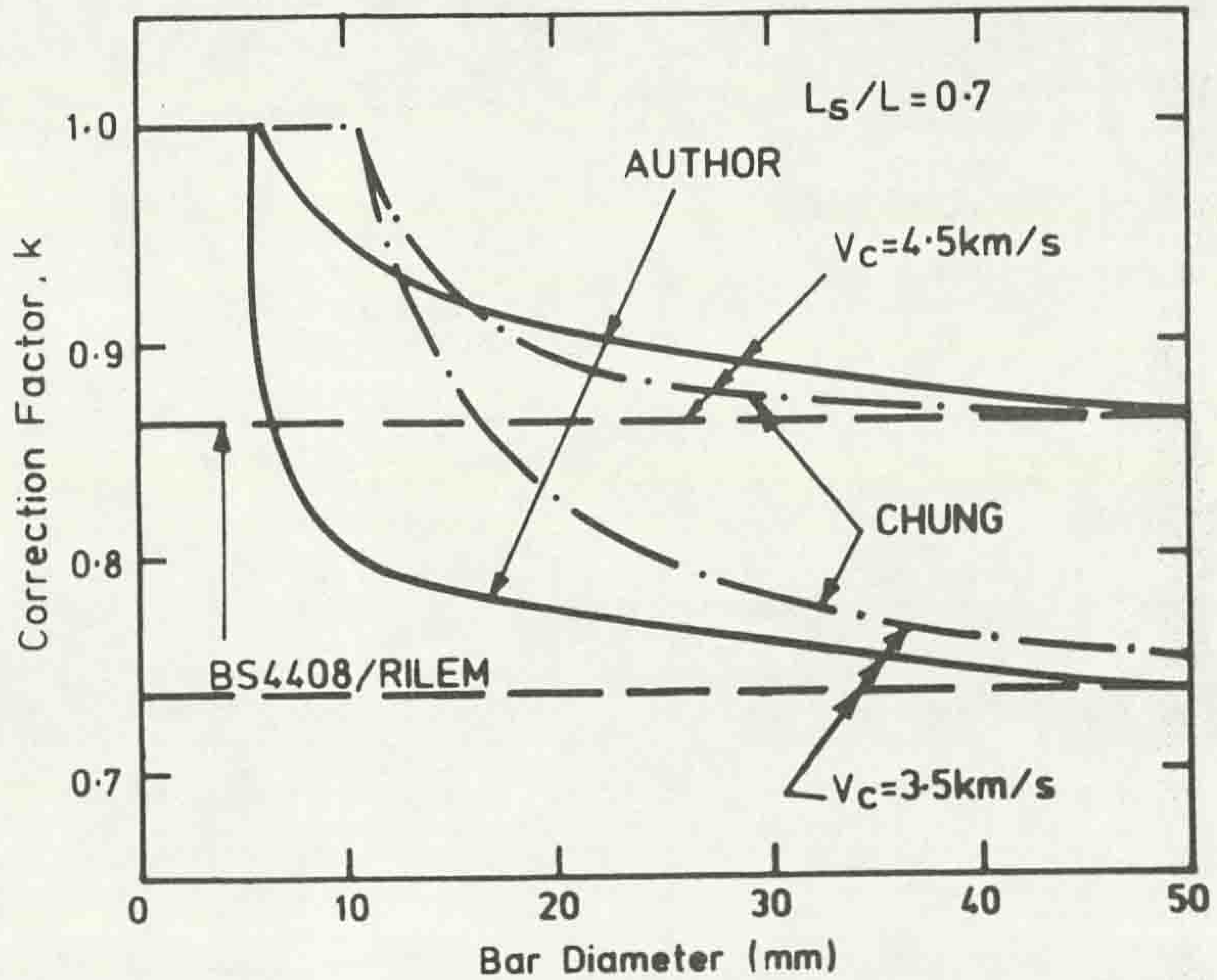

Figure 10. Comparison of Correction Procedures for Longitudinal Bars

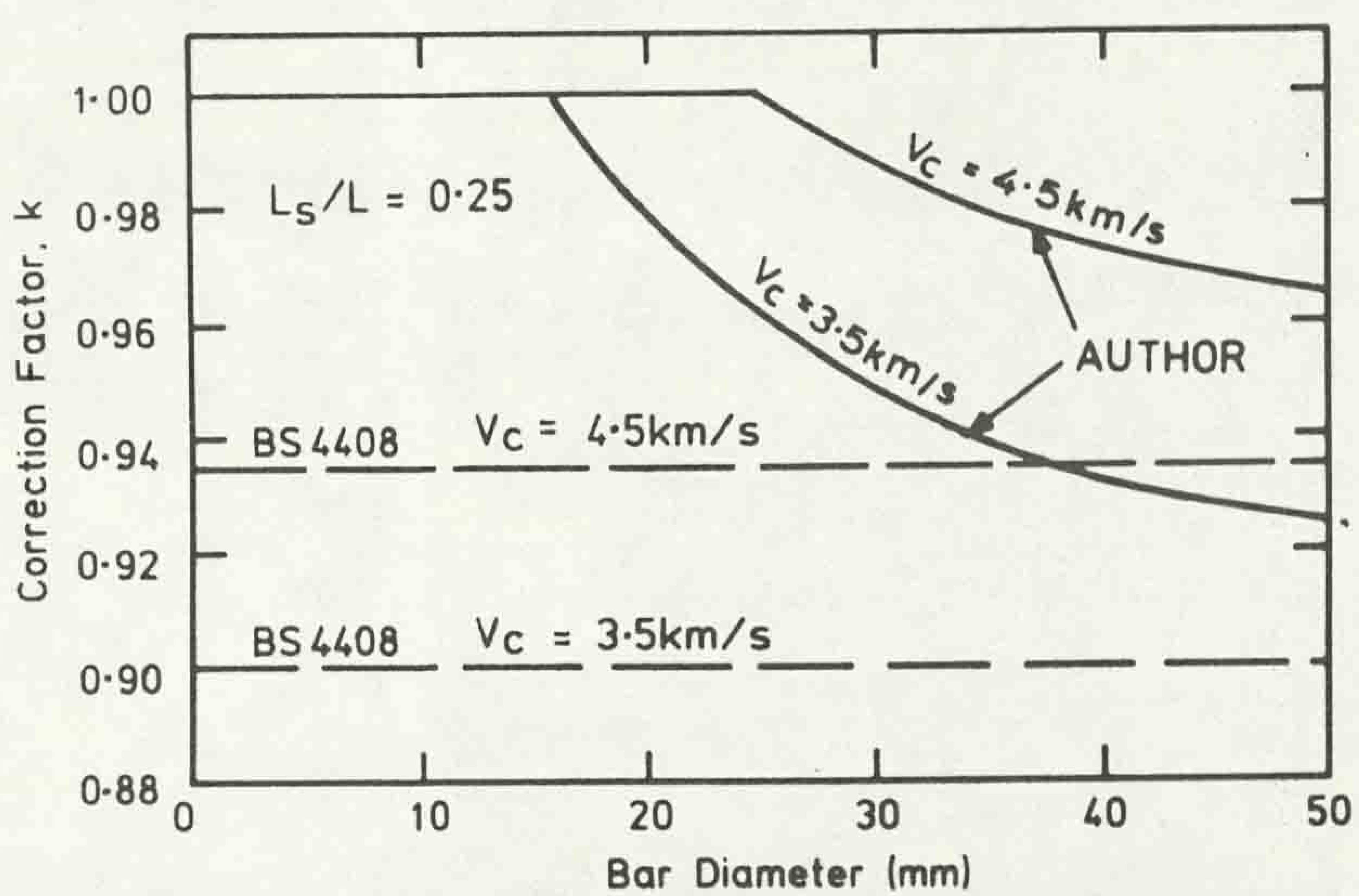

Figure 11. Comparison of Typical Correction Procedures for Transverse Bars.

Paper 10

"Non-destructive Testing-Developments in Test Methods"

Current Practice Sheet 88 Concrete

Vol. 17 No. 10 October 1983 pp.49-50

Current Practice Sheets

Test methods

Non-destructive testing: Part 2

Developments in test methods

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2P/20/2 No 88

CI/SfB/ / /q4/(Aq)
UDC 666.972.017: 620.179.1

Part 1, published in August, covered planning and interpretation of non-destructive testing.

The problems encountered with High Alumina Cement in the mid-1970s, an increasing interest in the economic advantages of early form stripping, and concern about durability have focussed attention upon the problems of in situ testing of concrete. As a result, the range of available non-destructive test methods has expanded considerably since the publication of Current Practice Sheet 10[3] in 1973. Ultrasonic pulse velocity and surface hardness were the principal methods available at that time, and their development in terms of technique, interpretation and applications has been continued[1]. Major developments of new tests have been concentrated in the areas of strength determination, integrity testing and potential durability assessment. Many of these new methods are fully established in the USA and other countries although they are not, as yet, covered by British Standards. BS4408[4] which deals with non-destructive test methods is, however, undergoing a major revision and it is likely that some of the new methods will be incorporated. These revisions will also reflect the developments which have taken place in the established testing methods.

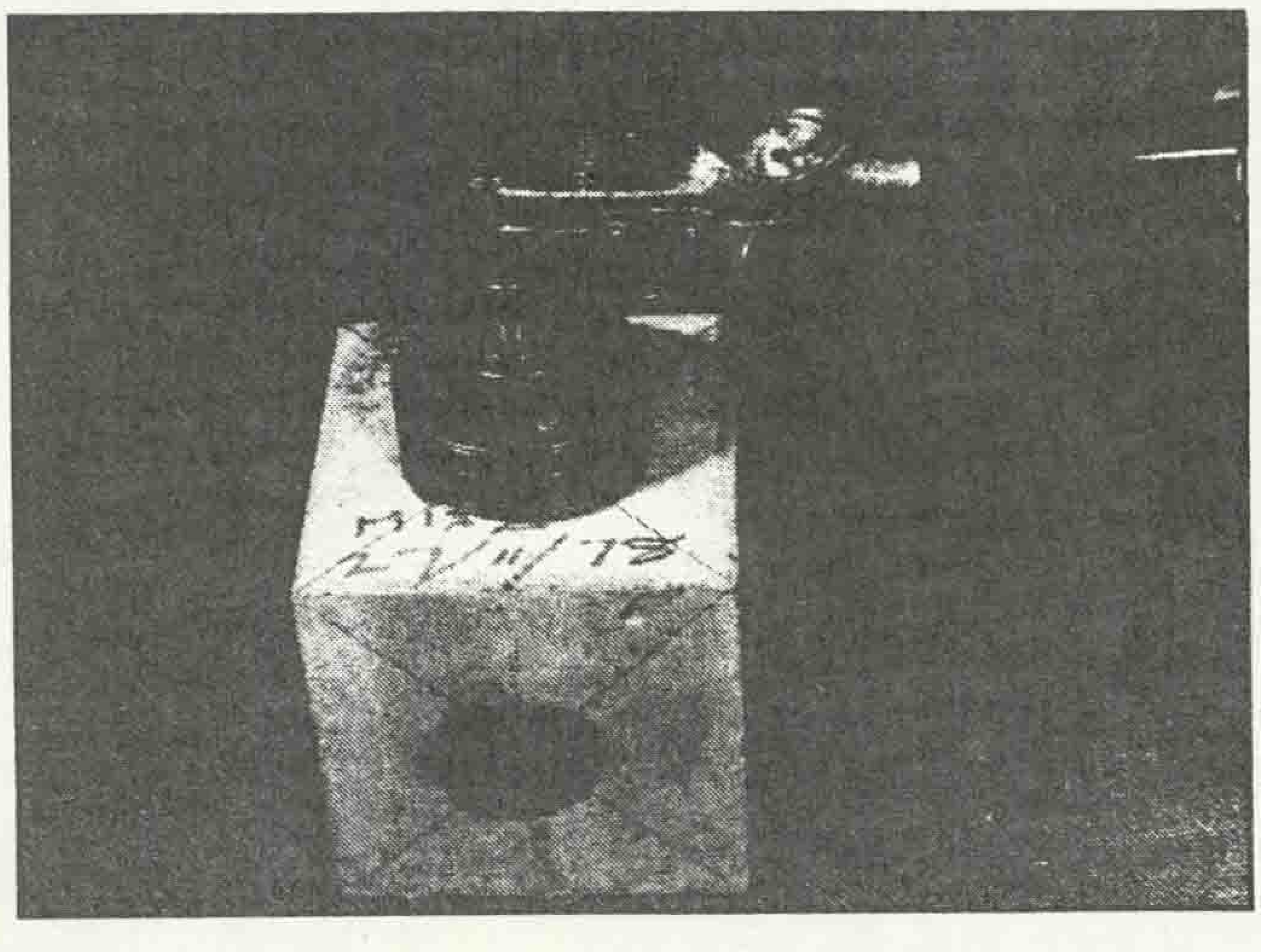
New strength assessment test methods

The principal new strength tests may be classified as 'near-to-surface' methods in that they only measure the concrete in the surface region. They also cause a limited amount of surface damage and may thus be classified as partially-destructive, although this damage is considerably less than that due to core cutting. These factors may obviously represent disadvantages, but these are offset by the important general characteristic that they directly measure a strength-related property. Calibrations are therefore not sensitive to such a wide range of variables as for the truly non-destructive test methods.

Internal fracture test

Developed at the Building Research Establishment in response to the need for a simple site test for strength measurement of High Alumina Cement Concrete, application of the method has since been extended to concrete made with Portland cement[5]. A 6mm diameter hole is drilled approximately 35mm deep into the concrete surface using a masonry drill. An expanding wedge anchor bolt is fixed into this hole to a depth of 20mm and pulled against an 80mm diameter reaction tripod by a torque-meter acting on a greased nut (Figure 2). The peak torque is observed and the average of six such values may be related to compressive strength with the aid of a calibration curve. Failure is initiated by internal fracturing of the concrete, but the load able to cause this is sensitive to loading technique, which must be carefully standardised. An important feature of this method is that a 'general' calibration curve may be used since sensitivity to aggregate characteristics and other features of the concrete mix is low.

Figure 2: Internal fracture test



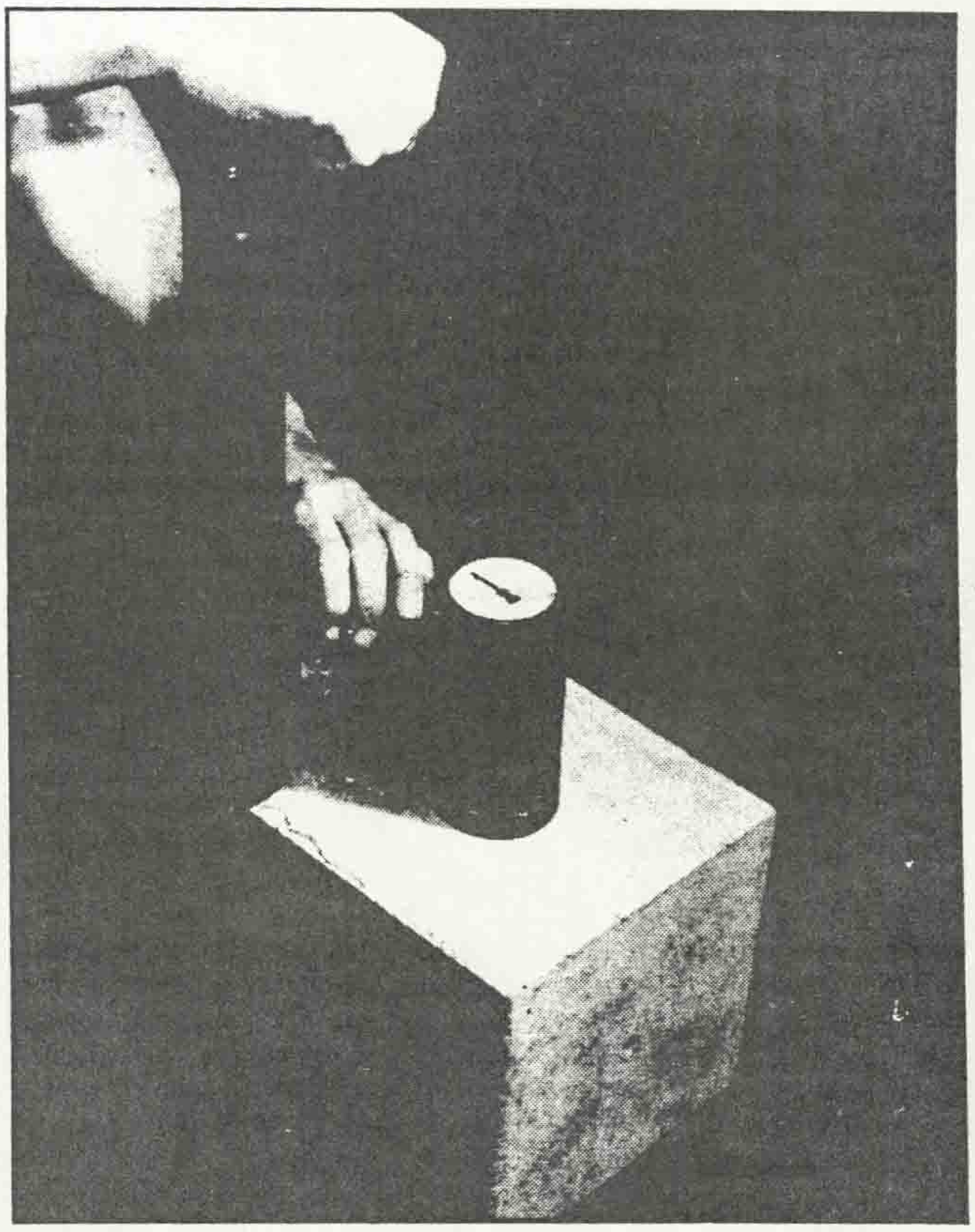


Figure 3: Lok-test

Pull-out tests

Developed principally in Denmark during the past 15 years these methods[6] are gaining acceptance in Scandinavia and the USA. A circular steel insert of 25mm diameter is located at a depth of 25mm below the concrete surface and pulled by means of a calibrated hand-operated hydraulic jack against a 55mm diameter reaction ring (Figure 3). The configuration is such that failure is due to compression of the concrete between the insert and reaction ring and thus relatively independent of other properties.

It is commercially available in two forms: The *Lok-test* which uses an insert cast into the concrete; and the *Capo-test* in which a steel ring is expanded into a groove undercut from an 18mm diameter drilled hole. The Lok-test is intended for quality control and strength development monitoring purposes, whilst the Capo-test is intended for strength assessment of existing concrete. In both cases, the average of six readings would normally be used for correlation against compressive strength using a linear relationship. For practical purposes this relationship may be regarded as 'general' in nature for concretes made with natural aggregates[1].

Pull-off test

A circular steel probe is bonded to the surface of the concrete by an epoxy resin adhesive. A specially designed portable apparatus is then used to pull off the probe, along with a bonded mass of concrete, by applying a direct tensile force (Figure 4). The peak load is measured, and as the failure area is approximately equal to that of the probe, a tensile strength can be calculated, and an equivalent cube strength estimated with the aid of a calibration graph. A modified version using partial coring can be used to overcome hard surface shell effects. Careful surface preparation is necessary but the results have been found to be consistent and reliable, and although the method is still under development at Queen's University, Belfast, a large scale in situ test programme has been undertaken with success[7].

Penetration resistance test

The most commonly known form of this approach[8] is the Windsor Probe Test in which a hardened steel alloy probe is fired into the concrete surface by a driver using a standardised powder cartridge (Figure 5). The depth of penetration, which will usually lie between 20mm and 40mm, is measured and the mean of three readings is related empirically to compressive strength by calibration charts. The principal factors influencing this relationship are aggregate hardness and type, and specific calibration is essential[1]. The method has been available in the USA for many years, and use in the UK has increased slowly. Although unsuitable for slender members the test is quick and useful where access may be difficult.

Break-off test

A circular core is formed or drilled in the concrete surface and the load required to break this off by means of a transverse force applied near the surface is measured. Developed in Norway, equipment is commercially available for this test and the method may be valuable as an in situ test for cases where tensile strength is important[9].

Integrity tests

Major developments have occurred in dynamic response testing enabling the dimensions and integrity of cast in situ piles to be quickly and reliably monitored. One popular approach is commonly known as the pulse-echo method[1], in which the reflected shock waves resulting from a single hammer blow on the top of the pile are measured with a hand held accelerometer and displayed on an oscilloscope. The impact tester developed by the Building Research Establishment to assess the integrity of screeds[10] also appears to have gained a considerable degree of acceptance. In this technique, the surface indentation resulting from a number of standard hammer blows is measured. Infra-red thermography is a further technique for assessment of structural integrity, which is under development in North America in relation to bridge decks, and is beginning to be used in this country for purposes such as the location of ducts and voids.

Potential durability assessment

Some of the established methods which are described in BS4408[4] and BS1881[12] such as Surface Hardness and Initial Surface Absorbtion may be used to give a comparative indication of concrete surface characteristics. Others, such as pulse velocity and radiographic surveys can detect further features affecting durability such as cracking and voids, whilst cover-measuring devices can confirm the thickness of protective concrete to reinforcing bars.

Simple radioactive and nuclear equipment has been developed by a number of manufacturers for the measurement of density and moisture condition in surface zones, and the depth of carbonation can be estimated with a simple chemical technique[13].

Newly developed non-destructive methods also relate to the electrical characteristics of the concrete and steel, and may be used to indicate the likelihood of reinforcement corrosion at any point[14]. These methods will not, however, either confirm that corrosion has actually occurred or assess its rate.

Half-cell potential assessments

The potential of embedded reinforcement[15] is measured relative to a copper/copper sulphate reference electrode placed on the concrete surface and connected to the bar through a high-impedance voltmeter. The concrete functions as an electrolyte, and the risk of corrosion may be related empirically to the measured potential difference.

Resistivity measurement

The resistivity of the surface zone concrete[15] is measured by a simple electrical technique involving four metal probes located on, or just below, the surface and coupled to a standard earth resistance meter. The value of concrete resistivity will control the magnitude of corrosion current that can flow, and hence provide an indication of the likelihood of corrosion in regions where a high risk has been identified by half-cell potentials.

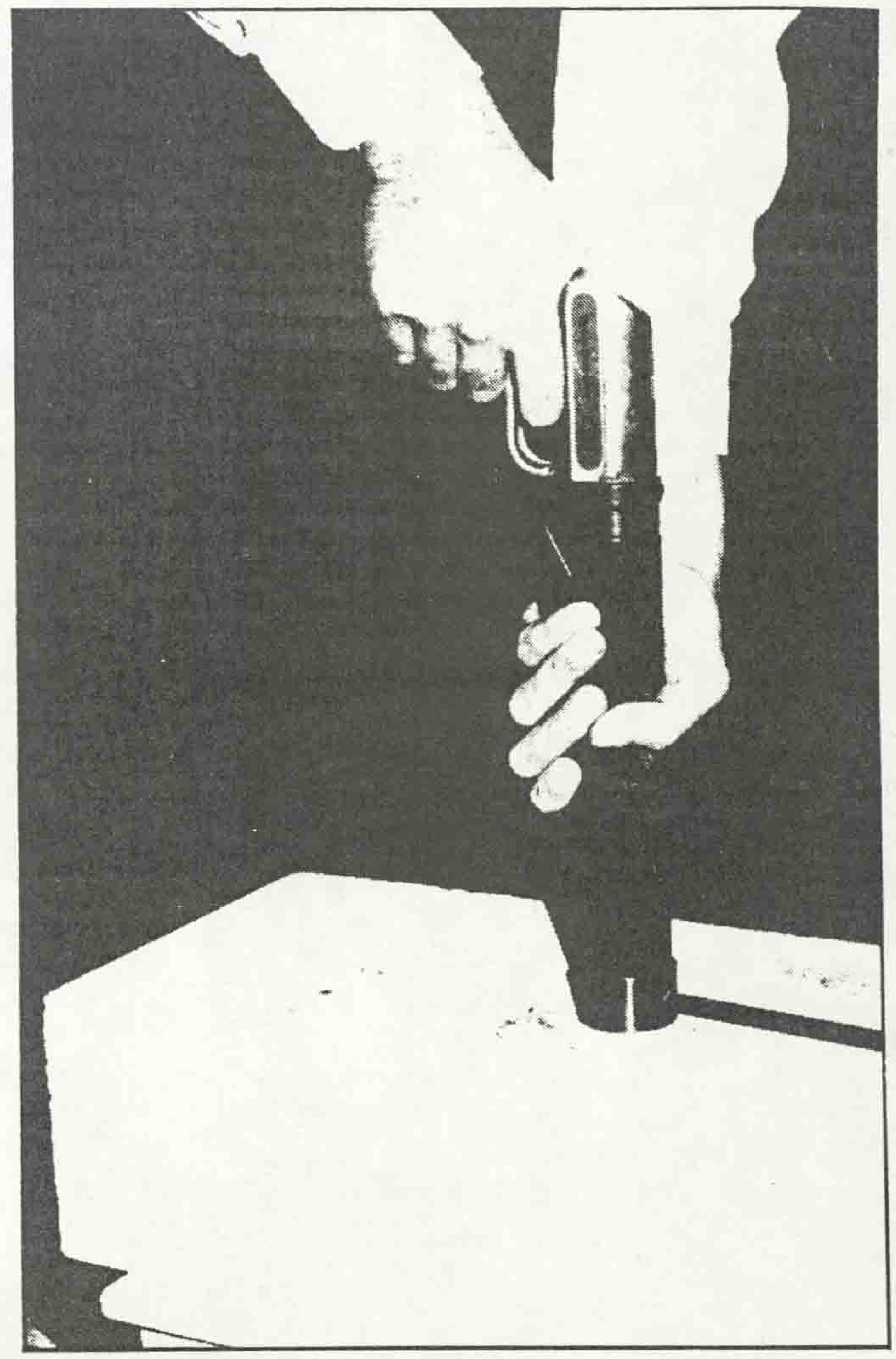
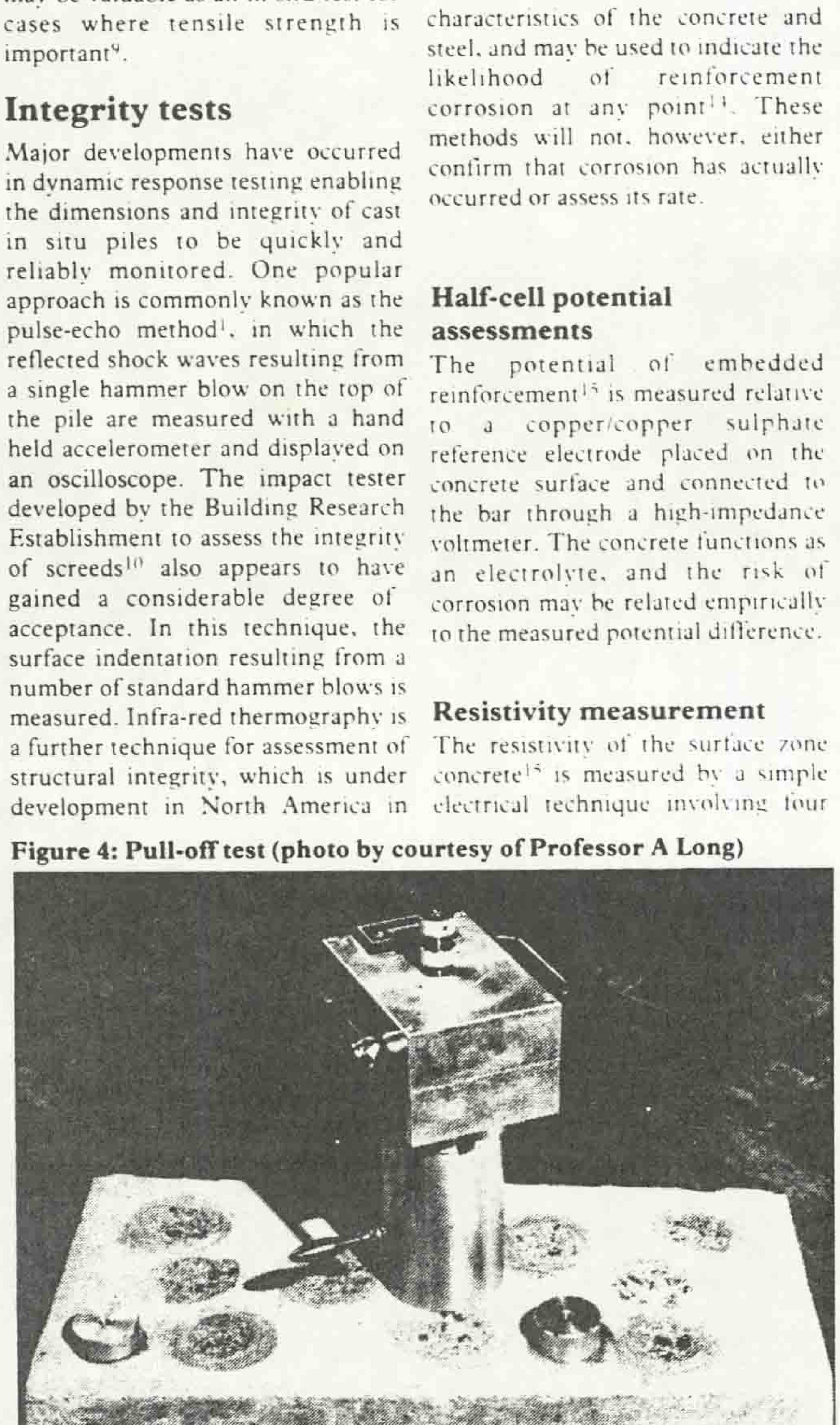


Figure 5: Windsor probe test

Figure 4: Pull-off test (photo by courtesy of Professor A Long)



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Paper 11

"Penetration Resistance, Pull-out, Pull-off and Break-off Methods"

The Testing of Concrete in Structures
Surrey University Press 1982
Chapter 4

4 Penetration resistance, pull-out, pull-off and break-off methods

Considerable developments have taken place in these forms of testing in recent years although at present they are not recognized as standard methods in the United Kingdom. All are surface zone semi-destructive methods which require only one exposed test surface. Unfortunately, all also have a number of serious drawbacks in application and accuracy. They should not, however, be discounted since there are many circumstances in which they have been shown to be of considerable value.

4.1 Penetration resistance testing

The technique of firing steel nails or bolts into a concrete surface to provide fixings is well established, and it is known that the depth of penetration is influenced by the strength of the concrete. A strength determination method based on this approach, using a specially designed bolt and standardized explosive cartridge, was developed in the USA during the mid 1960's and is known as the Windsor probe test (41). It is gaining popularity in the USA and Canada, especially for monitoring strength development on site, and is the subject of ASTM Standard C803-79 (42). Many authorities in North America regard it as equivalent to site cores, and in some cases it is accepted in lieu of control cylinders for compliance testing. Use outside North America has so far been limited, but the equipment is now more readily available and it seems likely that the method will become more widely used.

Although it is difficult to theoretically relate the depth of penetration of the bolt to the concrete strength, consistent empirical relationships can be found that are virtually unaffected by operator technique. The method is a form of hardness testing and the measurements will relate only to the quality of concrete near the surface, but it is claimed that it is the zone between approximately 25 and 75 mm below the surface which influences the penetration. This depth is considerably greater than for rebound or any other "surface zone" tests.

4.1.1 *Test equipment and operation*

The bolt or probe which is fired into the concrete (Figure 4.1) is of a hardened steel alloy. The principal features are a blunt conical end to punch through the matrix and aggregate near the surface, and a shoulder to improve adhesion to the compressed concrete and ensure a firm embedment.

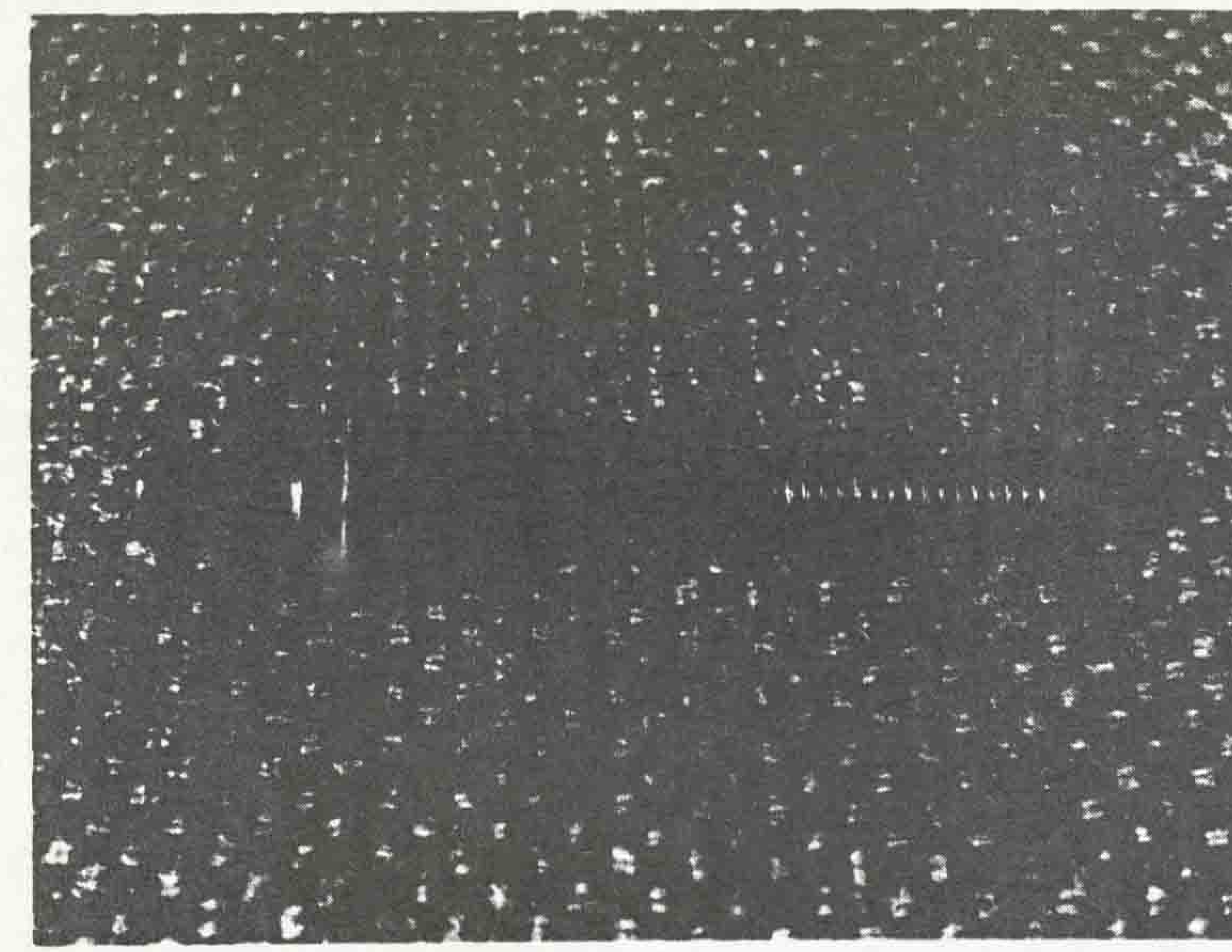

Figure 4.1 Penetration resistance test probe.

The probes are generally 6.35 mm in diameter and 79.5 mm in length, but larger diameter bolts (7.94 mm) are available for testing lightweight concretes. A steel firing head is screwed onto the threaded end of the bolt whilst the plastic guide locates the probe within the muzzle of the driver from which it is fired. The driver, which is shown in operation in Figure 4.2, utilizes a carefully standardized powder cartridge. This imparts a constant amount of energy to the probe irrespective of firing orientation, and produces a velocity of 183 m/sec which does not vary by more than ±1 %. The power level can be reduced when dealing with low strength concretes simply by locating the probe at a fixed position within the driver barrel. The driver is pressed firmly against a steel locating plate held on the surface of the concrete which releases a safety catch and permits firing when the trigger is pulled. After firing, the driver head and locating plate are removed and any surface debris around the probe is scraped or brushed away to give a level surface. A flat steel plate is placed on this surface, and a steel cap screwed onto the probe to enable the exposed

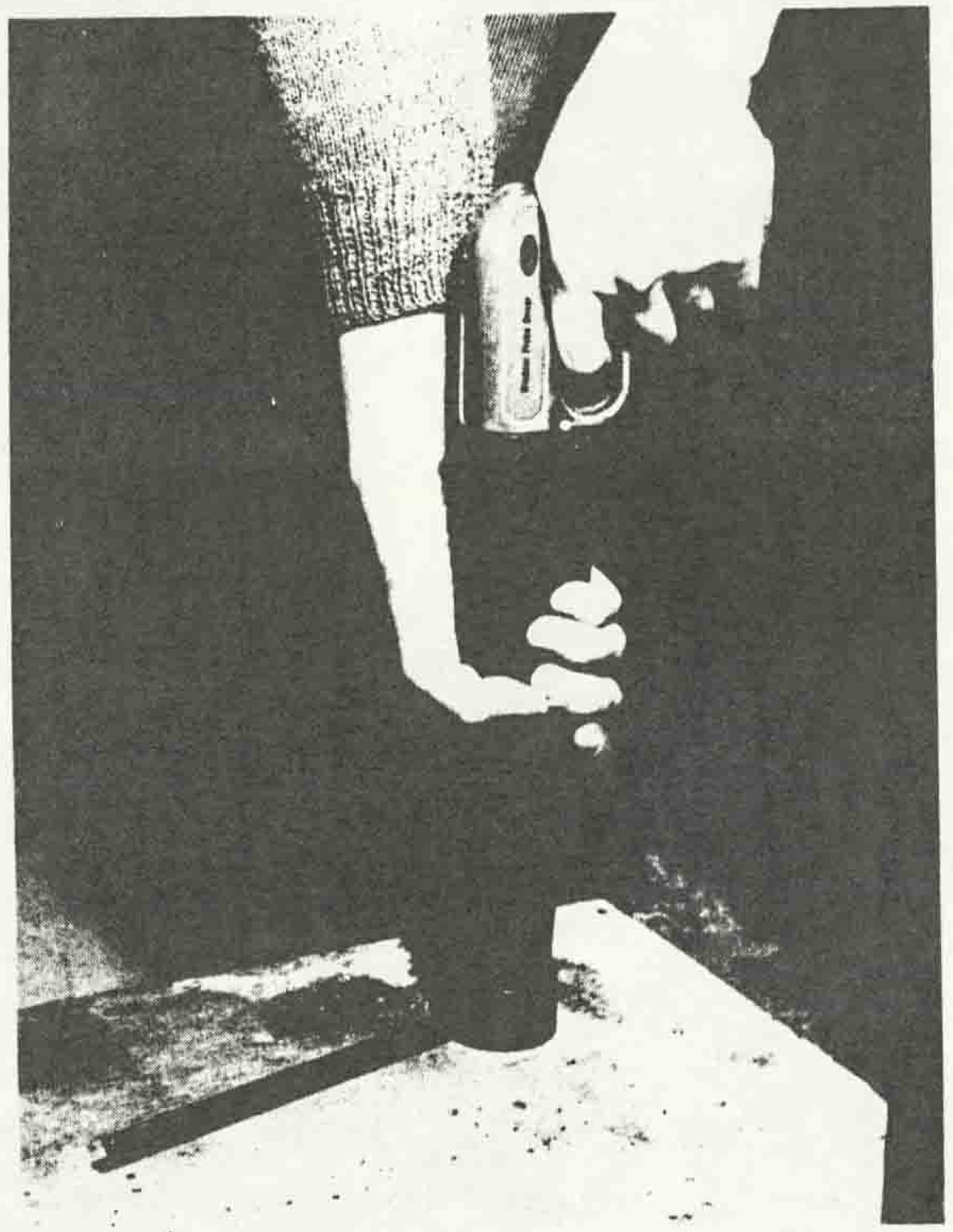

Figure 4.2 Driver in use.

length to be measured to the nearest 0.5 mm with a spring-loaded calibrated depth gauge, as in Figure 4.3.

Probe penetrations may be measured individually as described, or alternatively the probes may be measured in groups of three using a triangular template with the probes at 177 mm centres. In this case, a system of triangular measuring plates is used which will provide one averaged reading of exposed length for the group of probes. This approach may mask inconsistencies between individual probes, and it is preferable to measure each probe individually. The measured average value of exposed probe length may then be directly related to the concrete strength by means of appropriate calibration tables or charts.

4.1.2 *Procedure*

Individual probes may be affected by particularly strong aggregate particles near the surface, and it is thus recommended that at least three tests are made

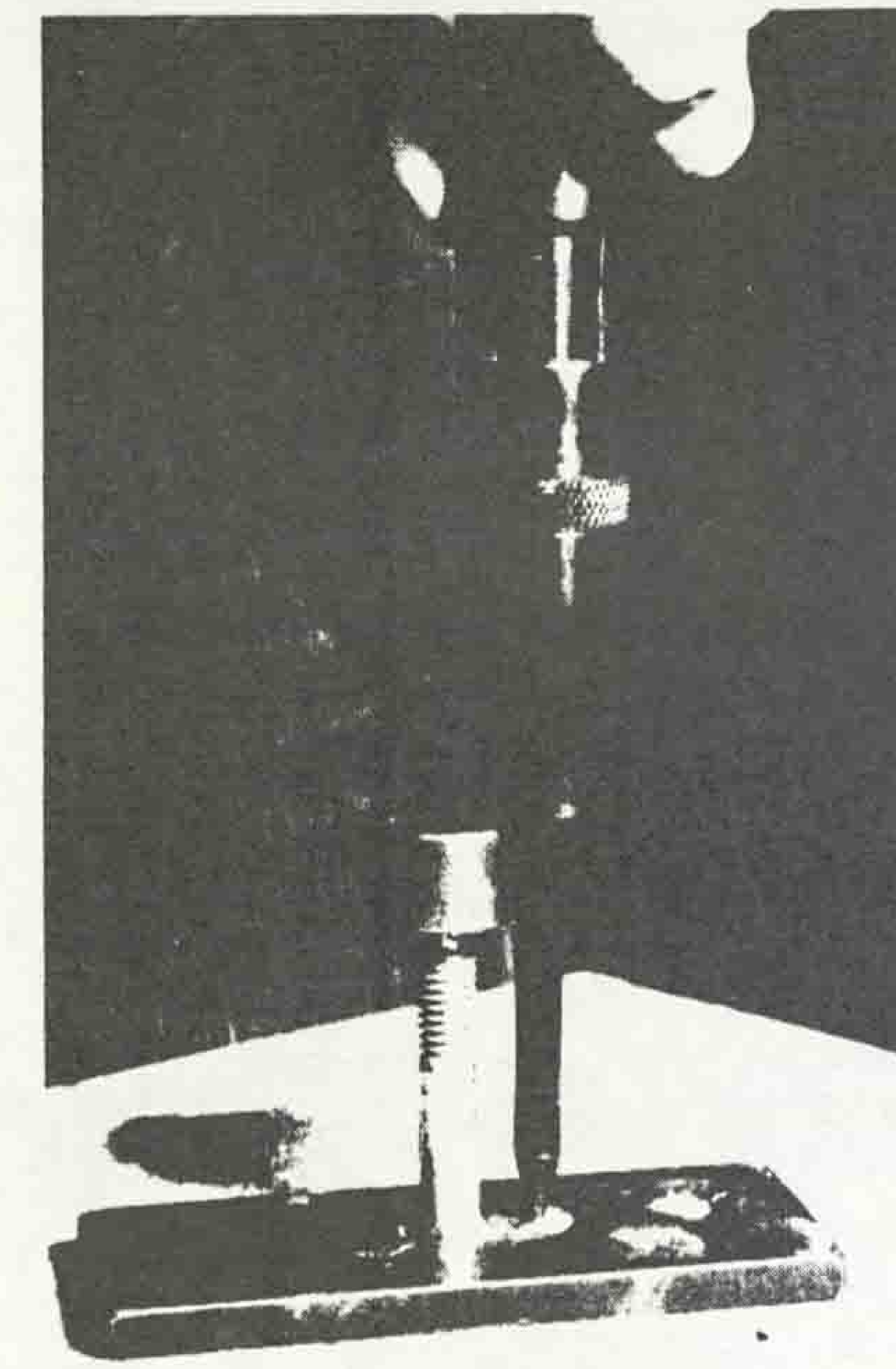

Figure 4.3 Height measurement.

and averaged to provide a result. If the range of a group of three tests exceeds 5 mm, a further test should be made and the extreme value discarded. Whilst slight surface roughness is not important, surfaces coarser than a broom finish should be ground smooth prior to test, and the probe must always be driven perpendicular to the surface.

Where the expected cube strength of the concrete is less than 26 N/mm^2, the "low power" setting should be used, but for higher strengths, this penetration may not be sufficient to ensure firm embedment of the probe, and the "standard power" setting is necessary. If probes will not remain fixed in very high strength concrete it may be possible to directly measure the depth of hole formed, after cleaning, and subtract this from the probe length. The manufacturers of the system recommend that a minimum edge distance of 100 mm should be maintained (75 mm for low power) but the author's experience suggests that these values may not always be sufficient to prevent splitting. Probes should also be at least 175 mm apart to avoid overlapping of zones of influence.

Aggregate hardness is an important factor in relating penetration to strength, and it may therefore be necessary to determine its value. This is assessed on the basis of the Mohs' hardness scale, which is a system for classifying minerals in terms of hardness into ten groups. Group 10 is the hardest and Group 1 the softest, thus any mineral will scratch another from a group lower than itself. Testing consists of scratching the surface of a typical aggregate particle with minerals of known hardness from a test kit; the hardest is used first, then the others in order of decreasing hardness until the scratch mark will wipe off. The first scratch that can be wiped off represents the Mohs' classification for the aggregate.

4.1.3 *Theory, calibration and interpretation*

4.1.3.1 Theory. A convincing theoretical description of the penetration of a concrete mass by a probe is not available since there is little doubt that a complex combination of compressive, tensile, shear and friction forces must exist. The manufacturers of the Windsor probe equipment have suggested that penetration is resisted by a subsurface compressive compaction bulb as shown in Figure 4.4. The surface concrete will crush under the tip of the probe, and the shock waves associated with the impact will cause fracture lines, and hence surface spalling, adjacent to the probe as it penetrates the body of concrete. The energy required to cause this spalling, or to break pieces of aggregate, is a low percentage of the total energy of a driven probe, and will therefore have a small effect upon the depth of penetration. Penetration will continue, with cracks not necessarily reaching the surface and eventually ceasing to form as the stress drops. Energy is absorbed by the continuous crushing at the point, surface friction and by compression of the bulb of contained concrete. It is this latter effect which prevents rebound of the probe, and it is claimed that the bulb, and depth of penetration, will be

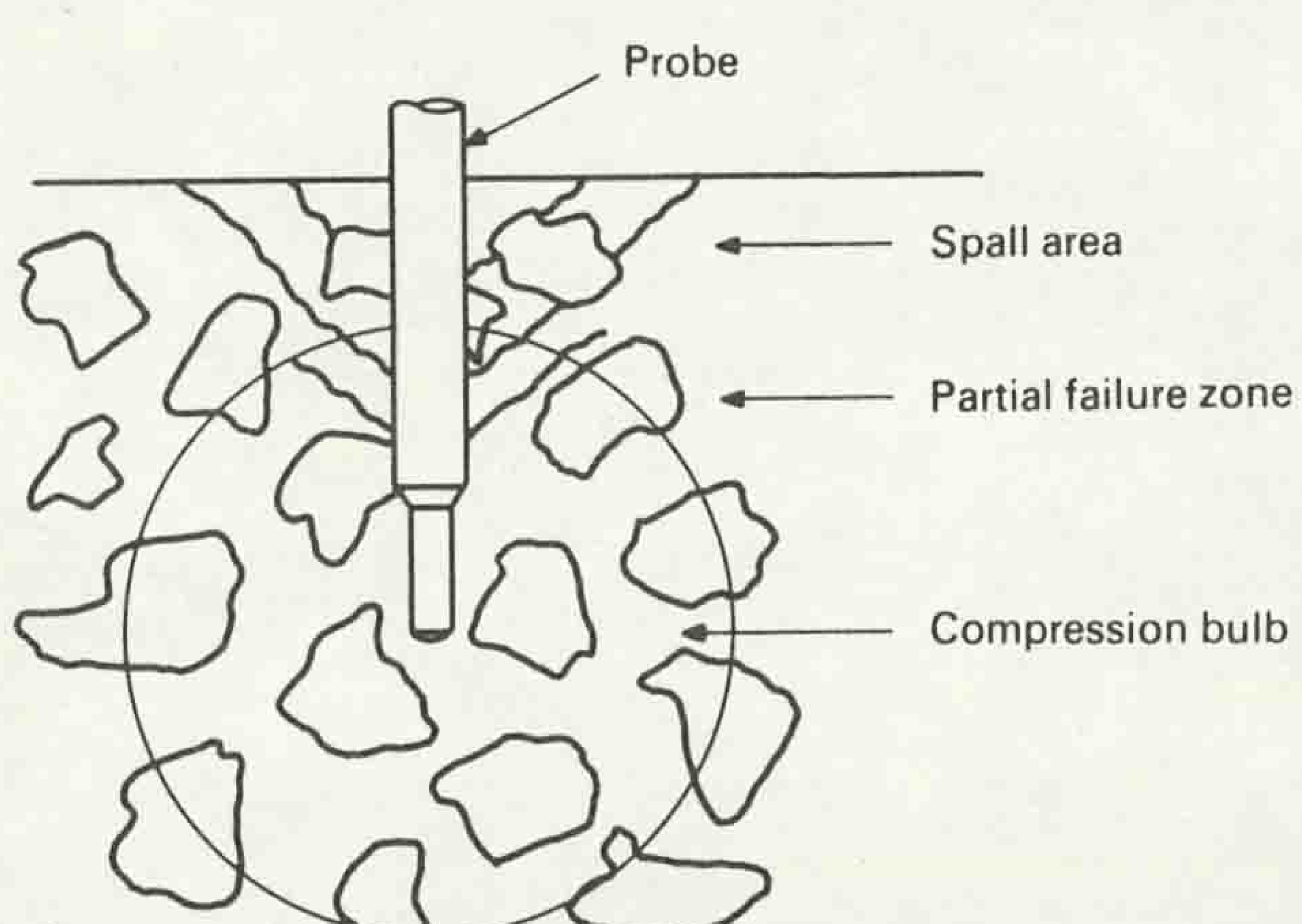

Figure 4.4 Compaction bulb.

inversely proportional to the compressive strength. Data is not currently available to support these proposals, which must be regarded as rather simplistic, although the concept of the measured property relating to concrete below rather than at the surface seems reasonable.

Whilst it may theoretically be possible to undertake calculations based on the absorption of the kinetic energy of the probe, this would be difficult, and it is very much easier to establish empirical relationships between penetration and strength.

4.1.3.2 Calibration. Calibration is hampered by the minimum edge distance requirement which prevents splitting. Although it may be possible to use standard 150 mm cubes or cylinders for tests at low power, the specimen must be securely held during the test. A holding jig for cylinders is available from the Windsor probe manufacturers, and cubes are most conveniently clamped in a compression testing machine, although no data concerning the influence of applied compressive strength is available. It is recommended by Malhotra (17) that groups of at least six specimens from the same batch are used, with three tested in compression and three each with one probe test, and the results averaged to produce one point on the calibration graph. Malhotra has also shown that the reduction in measured compressive strength of cylinders which have been previously probed may be up to 17.5%, and such specimens cannot therefore be tested in compression for calibration purposes.

Where the cube strength of the concrete is greater than 26 N/mm^2 it is necessary to use a combination of cubes or cylinders for compression testing and larger slab or beam specimens from the same batch for probing. The size of such specimens is unimportant provided that they are at least 200 mm in width and 150 mm in thickness, and preferably of sufficient length to accommodate at least three probes which satisfy the minimum edge distance and spacing requirements. These test specimens must be similarly compacted, and all should be cured together. In such situations, the use of ultrasonic pulse velocity measurements to compare concrete quality between specimens would be valuable. This approach has been used by the author (43) in an investigation in which 1000 × 250 × 150 mm beams were used for probing, and 100 mm cubes for compression testing, and it was found that the beam concrete was between 10 and 20% lower in strength than the concrete in the cubes. Since calibrations will normally relate to actual concrete strength it is also important that the moisture conditions of the specimens are similar. Figure 4.5 shows a typical calibration chart obtained in this way with a strength range obtained by water/cement ratio and age variations. Relationships between penetration and strength for the two different power levels are not easily related, and it is therefore necessary to produce calibration charts for each experimentally.

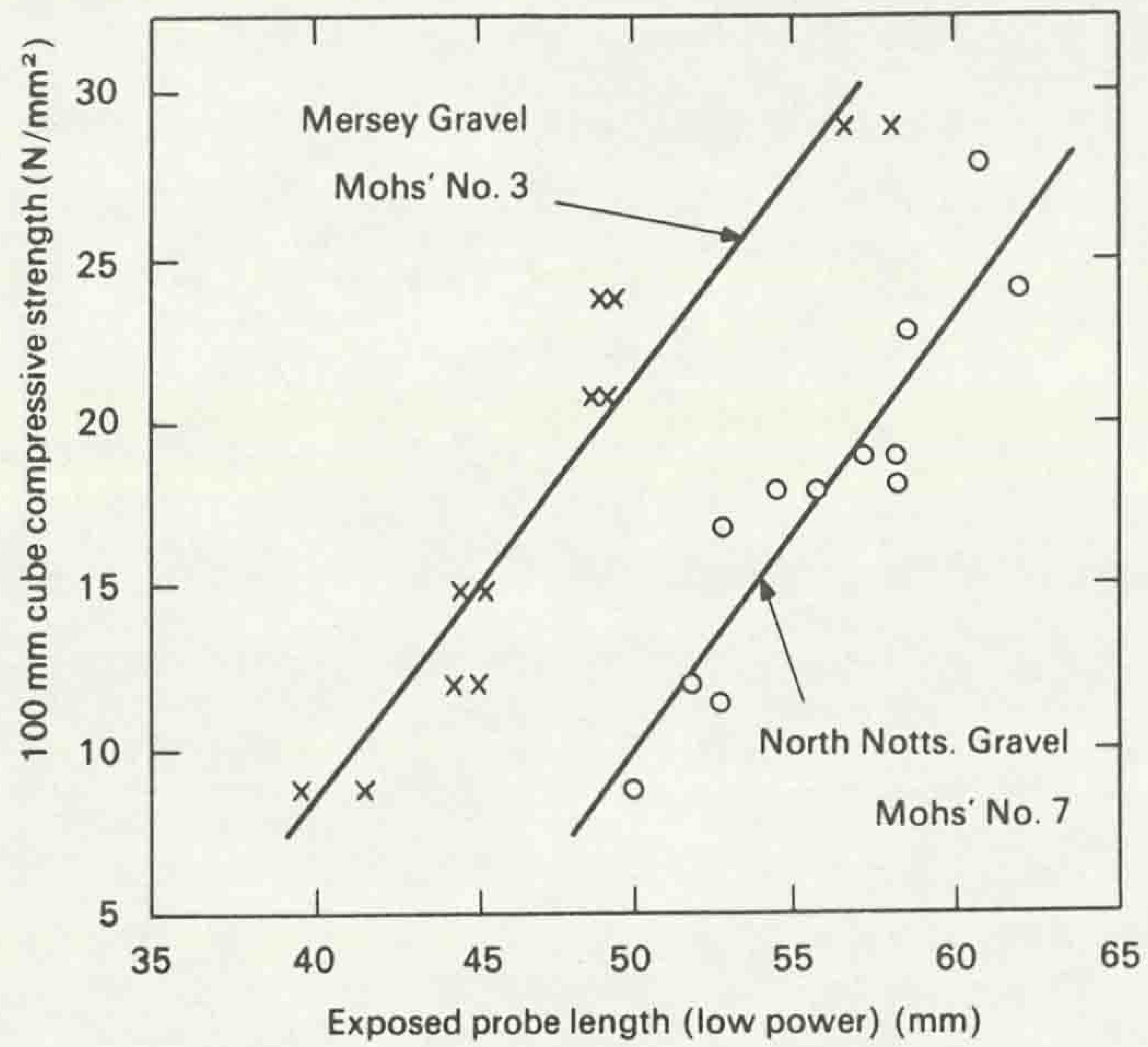

Figure 4.5 Typical low power strength calibration (based on ref. 43).

4.1.3.3 Interpretation. The manufacturers of the test equipment provide calibration tables (41) in which aggregate hardness is taken as the only variable influencing the penetration/strength relationship. It is clear from the

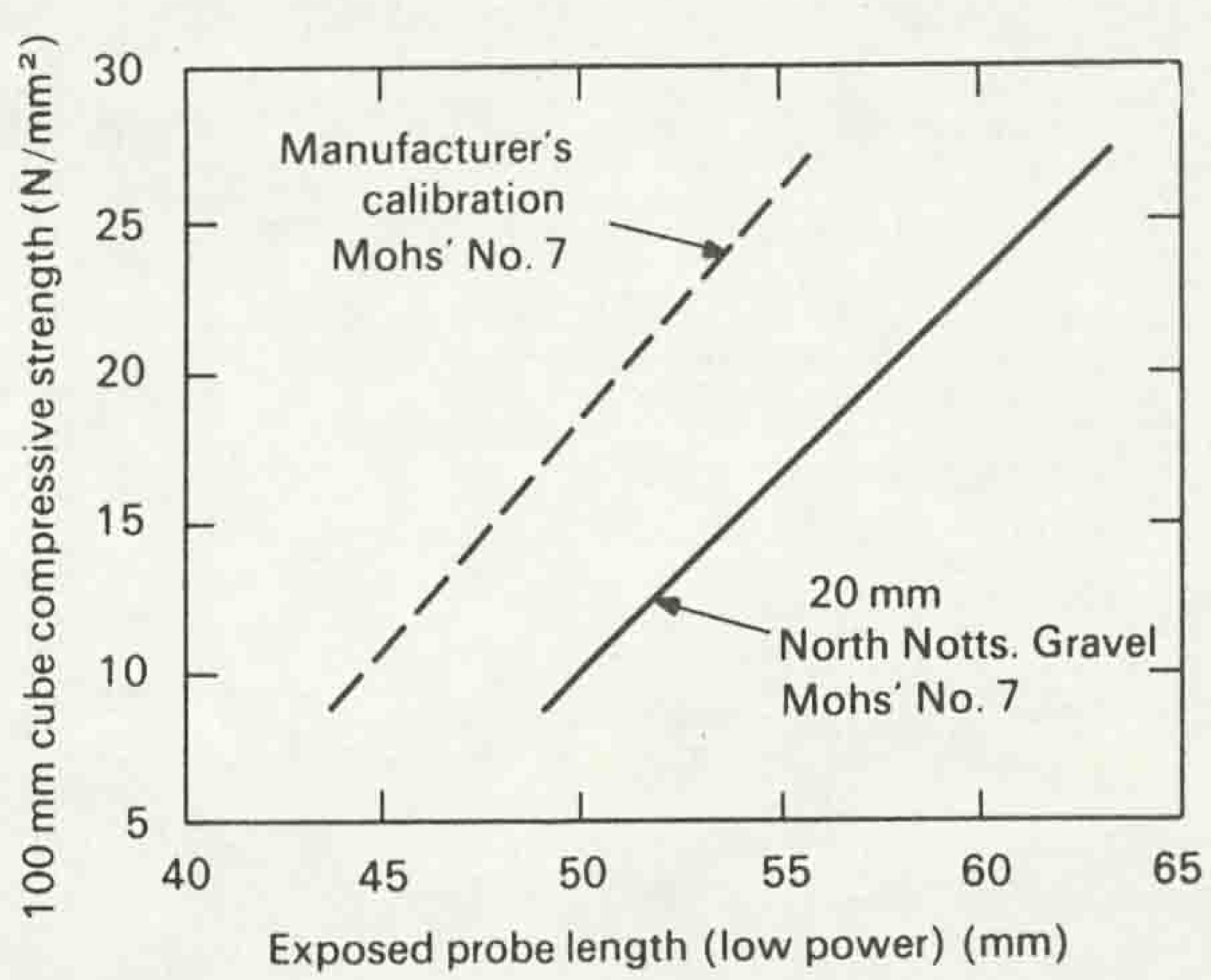

Figure 4.6 Comparison of calibrations (based on refs. 41 and 43).

author's work (43) and from reported experience in the USA (17) that this is not the case, and that aggregate type can also have a large influence. It is understood that the manufacturer's tables are based on crushed rock, but for rounded gravels the crushing strength may be lower than suggested by probe results. It is to be expected that bond differences at the aggregate/matrix interface due to aggregate surface characteristics may affect penetration resistance and crushing strength. Nevertheless, the extent of the calibration discrepancy which may be attributed to this, as indicated in Figure 4.6, is disturbing.

Not many data are available for this test, but calibrations from a number of sources are compared in Figure 4.7. It appears that moisture condition,

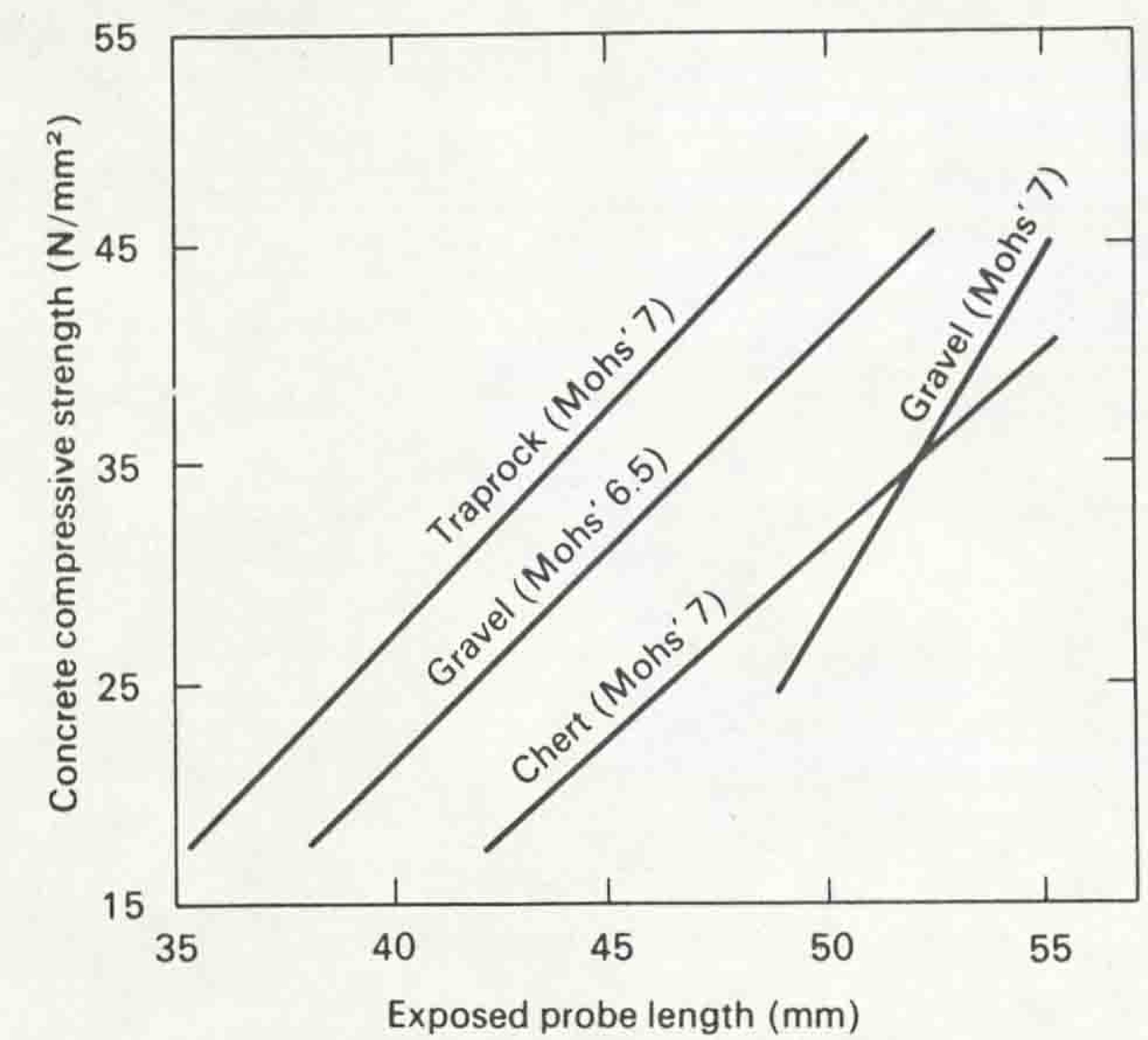

Figure 4.7 Influence of aggregate type and proportions (based on ref. 17).

aggregate size (up to 50 mm), aggregate proportions and age all have effects which are small in relation to aggregate hardness and type. It is essential therefore that calibration charts should be developed for the particular aggregate type involved in any practical application of the method.

4.1.4 *Reliability, limitations and applications*

4.1.4.1 Reliability. The test is not greatly affected by operator technique, although verticality of the bolt relative to the surface is obviously important and a safety device in the driver prevents firing if alignment is poor. It is claimed (44) that an average coefficient of variation for a series of groups of three readings on similar concrete of the order of 4% may be expected, and that a correlation coefficient of greater than 0.98 can be achieved for a linear calibration relationship for a single mix. Field tests by the author on motorway deck slabs have also yielded a similar coefficient of variation of probe results over areas involving several truck loads of concrete. It is also apparent from Figure 4.5 that 95% limits of about $\pm 20\%$ on predicted strengths may be possible for a single set of three probes, given adequate calibration charts. It is to be expected that aggregate size will influence the scatter of individual probe readings, but at present insufficient data is available to assess the effect of this on strength prediction accuracies, although a 50 mm maximum size is recommended. Similarly, the effect of reinforcement adjacent to the probe is uncertain, and a minimum clearance of 50 mm should be allowed between probes and reinforcing bars.

4.1.4.2 Limitations. The principal physical limitation of this method is caused by the need for adequate edge distances and probe spacings together with a member thickness of at least twice the anticipated penetration. After measurement the probe can be extracted, leaving a conical damage zone (Figure 4.8) which must be made good. There is the additional danger of splitting of the member if it does not comply with the minimum recommended dimensions. Expense is a further consideration, since at 1981 prices, the equipment cost in the United Kingdom is approximately £900 with a recurrent cost of £3.50 per set of 3 probes.

4.1.4.3 Applications. The limitations outlined above mean that although probe measurement takes place at a greater depth within the concrete than rebound hammer measurements, penetration tests are unlikely to replace rebound tests except where the latter are clearly unsatisfactory. Probes cannot examine the interior of a member in the same way as ultrasonics and the method causes damage that must be repaired. However, probes do offer the advantage of requiring only one surface and fewer calibration variables. In relation to cores, however, probes provide easier testing methods, speedy results and accuracy of strength estimation comparable to small diameter specimens. Although the accuracies of large-diameter cores cannot be matched, it is likely that probing may be used as an alternative to cores in some circumstances, and a case history of this nature has been reported (45).

In the USA there is a trend towards in-place compliance testing, especially in relation to post-tensioning. Since the most reliable application of the penetration method lies in comparison of similar concrete where specific calibration charts can be obtained, a number of applications of this nature

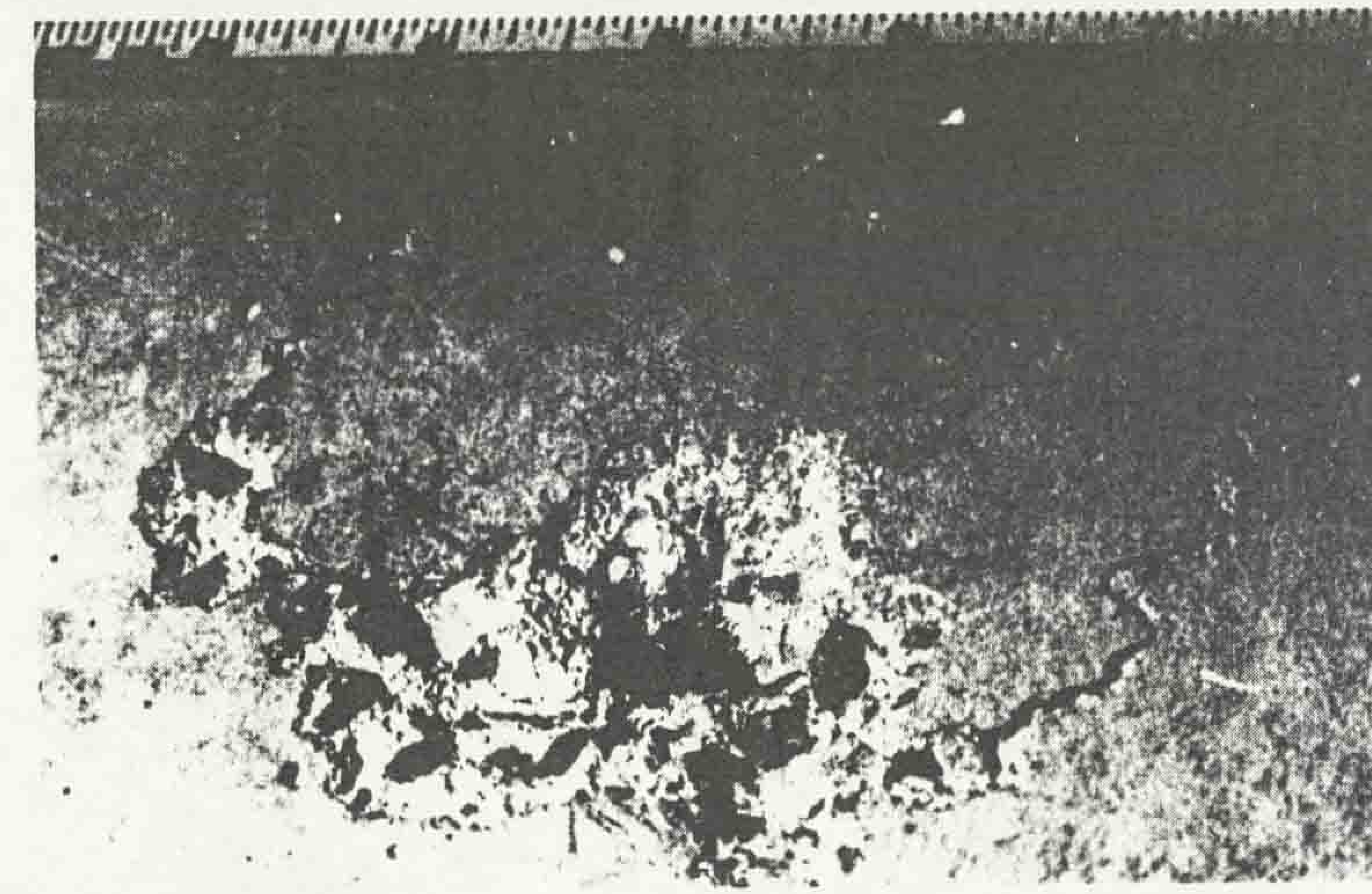

Figure 4.8 Surface damage caused by probe removal.

have been reported (17, 46). It appears that the advantages of speed and simplicity, together with the ability to drive probes through timber or even thin steel formwork without influence, outweigh the cost. Details of acceptable thicknesses are unfortunately not available, but in such circumstances decisions would normally be based on previously established "go/no go" limits for measured penetration.

Other applications include the detection of substandard members or areas of mature concrete, and this method is particularly appropriate for large walls or slabs having only one exposed surface free of finishes. Investigations of this type have been successfully performed by the author on highway bridge deck slabs. Probing was carried out on the deck soffit from a small mobile hydraulic platform whilst the road above was in normal use, and in one such investigation a total of 18 sets of probes were placed by one operator in a period of six hours. The speed of operation, together with the immediate availability of results, means that many more tests can be made than if cores were being taken, and test locations can be determined in the light of the results obtained. This is particularly valuable when attempting to define the location and extent of substandard concrete.

Whether or not the test will be of significant value in the strength assessment of "unknown" concrete is uncertain, but it is clear that results based solely on aggregate hardness are inadequate. It may be, however, that as more results are made available it will be possible to increase the confidence with which the method may be extended beyond comparative situations.

4.2 Pull-out testing

The concept of measuring the force needed to pull a bolt or some similar device from a concrete surface has been under examination for many years. Proposed tests fall into two basic categories; those which involve an insert which is cast into the concrete, and those which offer the greater flexibility of an insert fixed into a hole drilled into the hardened concrete. Cast-in methods must be pre-planned and will thus be of value only in testing for specification compliance, whilst drilled hole methods will be more appropriate for field surveys of mature concrete. In both cases, the value of the test depends upon the ability to relate pull-out forces to concrete strengths and a particularly valuable feature is that this relationship is relatively unaffected by mix characteristics and curing history. Although the results will relate to the surface zone only, the approach offers the advantage of providing a more direct measure of strength and at a greater depth than surface hardness testing by rebound methods, but still requires only one exposed surface.

4.2.1 *Cast-in methods*

Reports were first published in the USA and USSR in the late 1930's describing tests in which a cast-in bolt is pulled from the concrete. These methods do not appear to have become popular, and it was not until 30 years later that practically feasible tests were developed. Two basic methods, both of which require a threaded insert which is fixed to the shuttering prior to concreting, have emerged. A bolt is then screwed into the insert and pulled hydraulically against a circular reaction ring. The principal difference

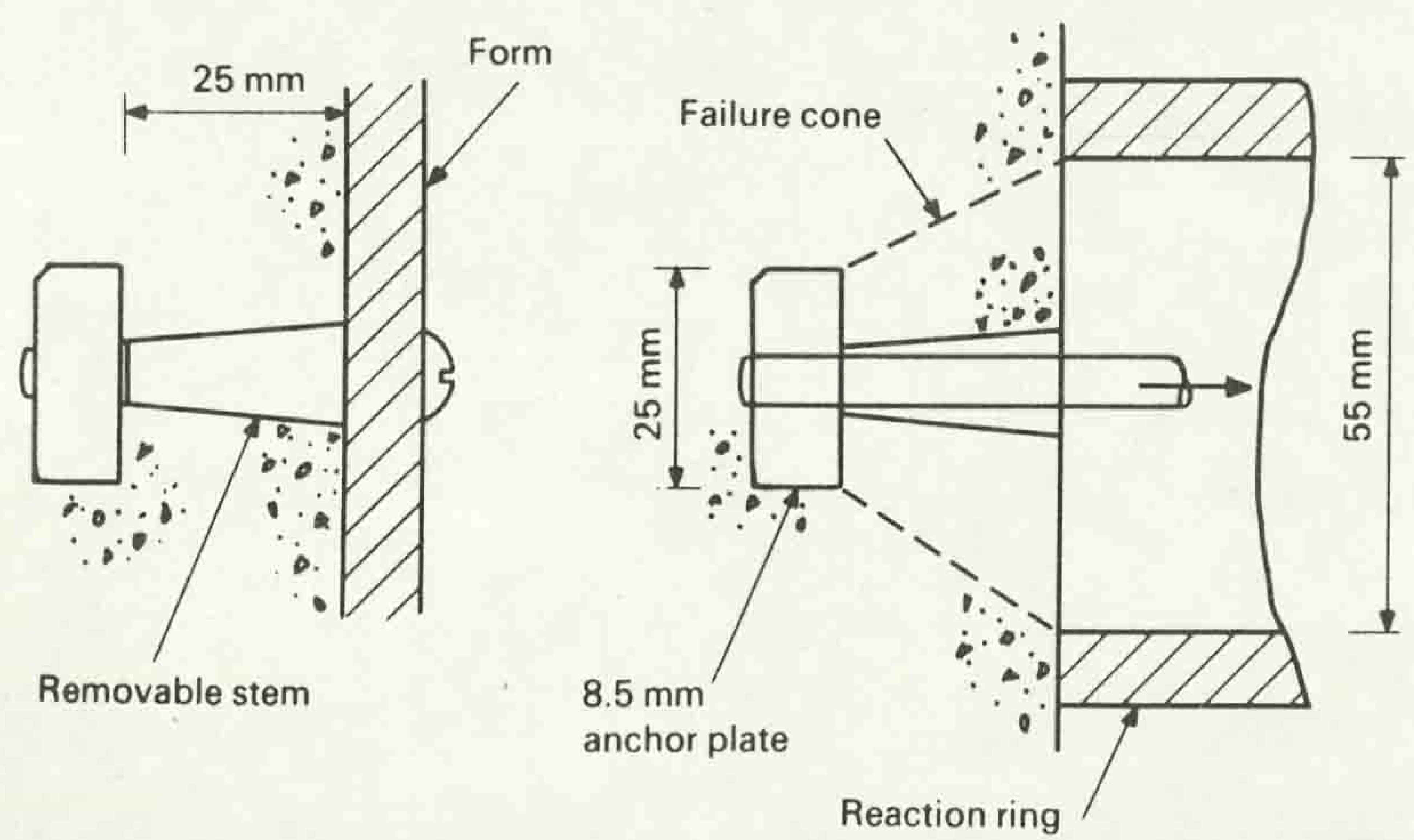

Figure 4.9 Lok-test insert.

between the two systems, developed in Denmark and Canada respectively, lies in the shape of insert and loading technique. In both cases a cone of concrete is "pulled-out" with the bolt, and the force required to achieve this is translated to compressive strength by the use of an empirical calibration.

4.2.1.1 The Lok-test. This approach, developed at the Danish Technical University (47) in the late 1960's, has gained popularity in Scandinavia and is now accepted by a number of public agencies in Denmark as equivalent to cylinders for acceptance testing.

The insert (Figure 4.9) consists of a steel sleeve which is attached to a 25 mm diameter, 8 mm thick anchor plate located at a depth of 25 mm below the concrete surface (48). The sleeve is normally screwed to the shuttering, or fixed to a plastic buoyancy cup where slabs are to be tested. This is later removed and replaced by a 7·2 mm diameter rod which is screwed into the

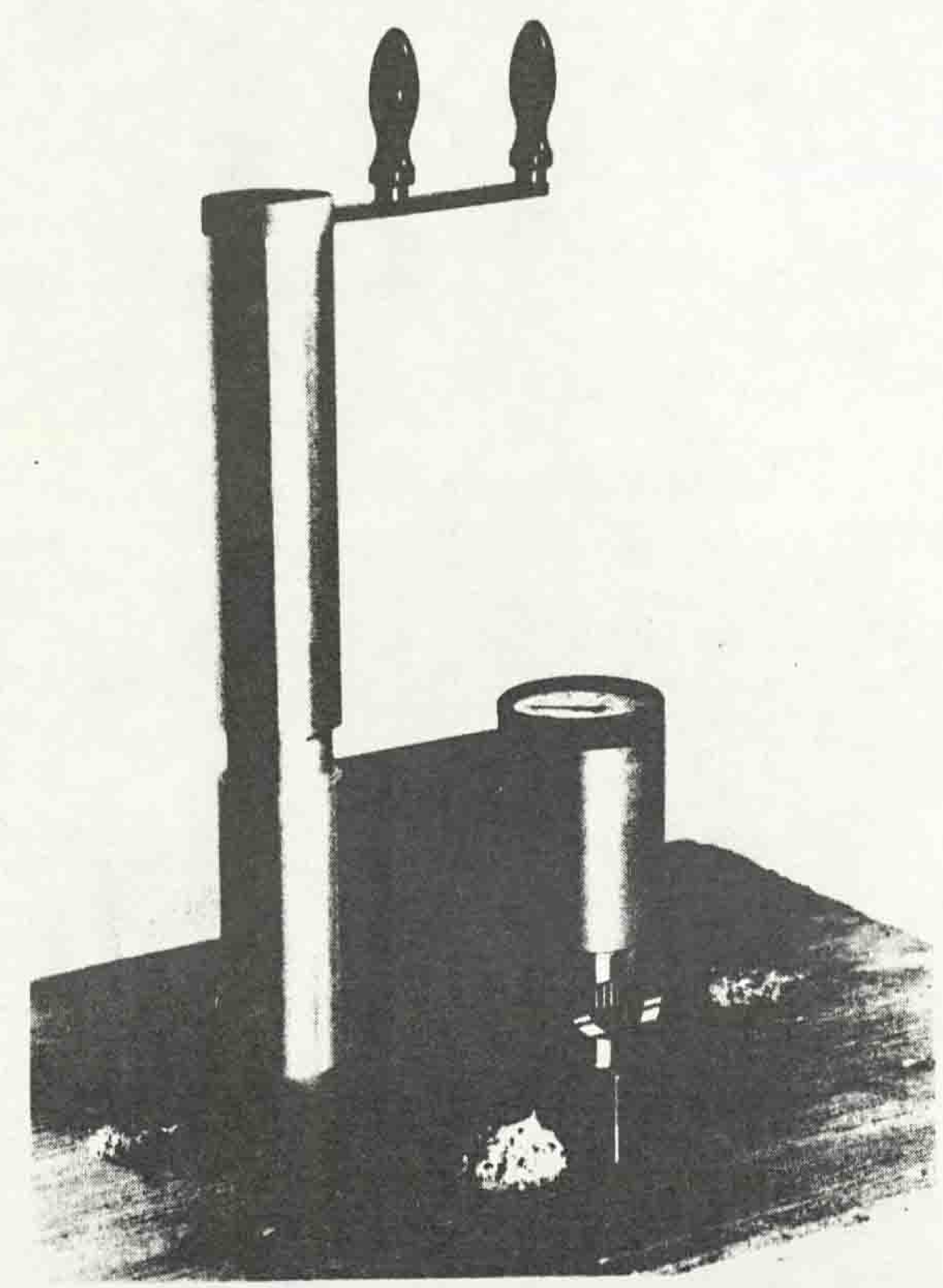

Figure 4.10 Lok-test equipment (photograph by courtesy of Lok-test Aps.).

anchor plate and coupled to a tension jack. The whole assembly is coated to prevent bonding to the concrete, and rotation of the plate is prevented by the "cut-off". Load is applied to the pull-bolt by means of a portable hand-operated hydraulic jack with a reaction ring of 55 mm diameter. This equipment (Figure 4.10) is compact, with a weight of less than 5 kg.

The loading equipment can determine the force required to cause failure by pulling the disc, and it is claimed that this will cover concrete with cube strengths in the range 8–69 N/mm^2. The load is measured with an accuracy of $\pm 2\%$ over normal operating temperatures, and a precision valve system combined with a friction coupling ensures a constant loading rate of 30 ± 10 kN/min. Load is released as soon as a peak is reached, leaving only a fine circular crack on the concrete surface. Calibration charts and tables provided by the manufacturer (Figure 4.11) are then used to estimate the compressive strength of the concrete.

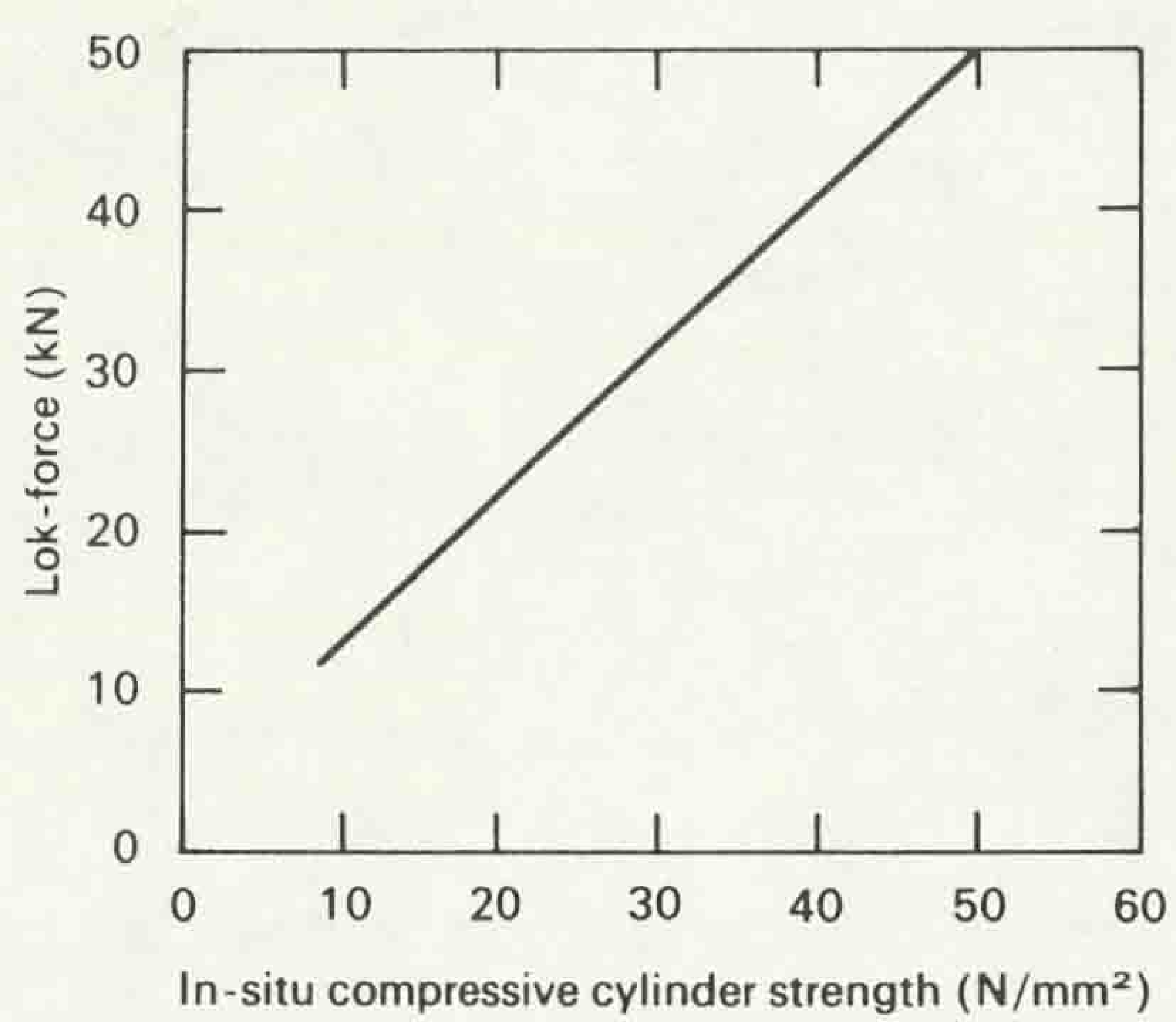

Figure 4.11 Typical Lok-test calibration chart (based on ref. 51).

The geometric configuration indicated in Figure 4.9 ensures that the failure surface is conical and at an angle of approximately 31° to the line of pull. This is close to the angle of friction of concrete, which is generally assumed to be 37°, and extensive theoretical work (49) has shown that this produces the most reliable measure of compressive strength. Plasticity theory for concrete using a modified Coulomb's failure criterion indicates that where the failure angle and friction angle are equal, the pull-out force is proportional to compressive strength. This approach has been used to confirm the experimentally determined relationships. Recent finite element analyses of the failure mechanism (50) have also indicated that failure is initiated by

crushing, rather than cracking, of the concrete. It is suggested that a narrow symmetrical band of compressive forces run between the cast-in disc and the reaction tube on the surface, and this further confirms that compressive strength is the main influence on pull-out force using this configuration.

The reliability of the method is reported to be good, with correlation coefficients for laboratory calibrations of about 0.96 on straight line relationships, and a corresponding coefficient of variation of about 7%. Comparison with rebound hammer and ultrasonic pulse velocity strength calibrations shows that the slope is much steeper, hence this test is much more sensitive to strength variations (51). An important feature of this approach is the independence of the calibration of features such as water/cement ratio, curing, cement type and aggregate properties (up to 38 mm maximum size). Strength calibration is thus more dependable than for most other non-destructive or semi-destructive methods. However, for large projects it is recommended that a specific calibration is developed for the concrete actually to be used. It should also be noted that artificial lightweight aggregates are likely to require specific calibration. The two principal limitations are preplanned usage (although the recently developed Capo test, section 4.2.2.2, overcomes this), and the surface zone nature of the test. The test equipment can be obtained in a convenient briefcase kit form containing all the necessary ancillary items, although the cost is relatively high: the cheapest kit was priced at over £1000 in 1981.

Bickley (52) indicates that the use of this approach is growing rapidly in North America, especially for determination of form stripping times in cooling tower construction. There seem to be few practical problems associated with in-situ usage, and an arrangement such as that shown in Figure 4.12 may be convenient in this situation. Other applications include determination of stressing times in post-tensioned construction, whilst in Denmark the approach is accepted as a standard in-situ strength

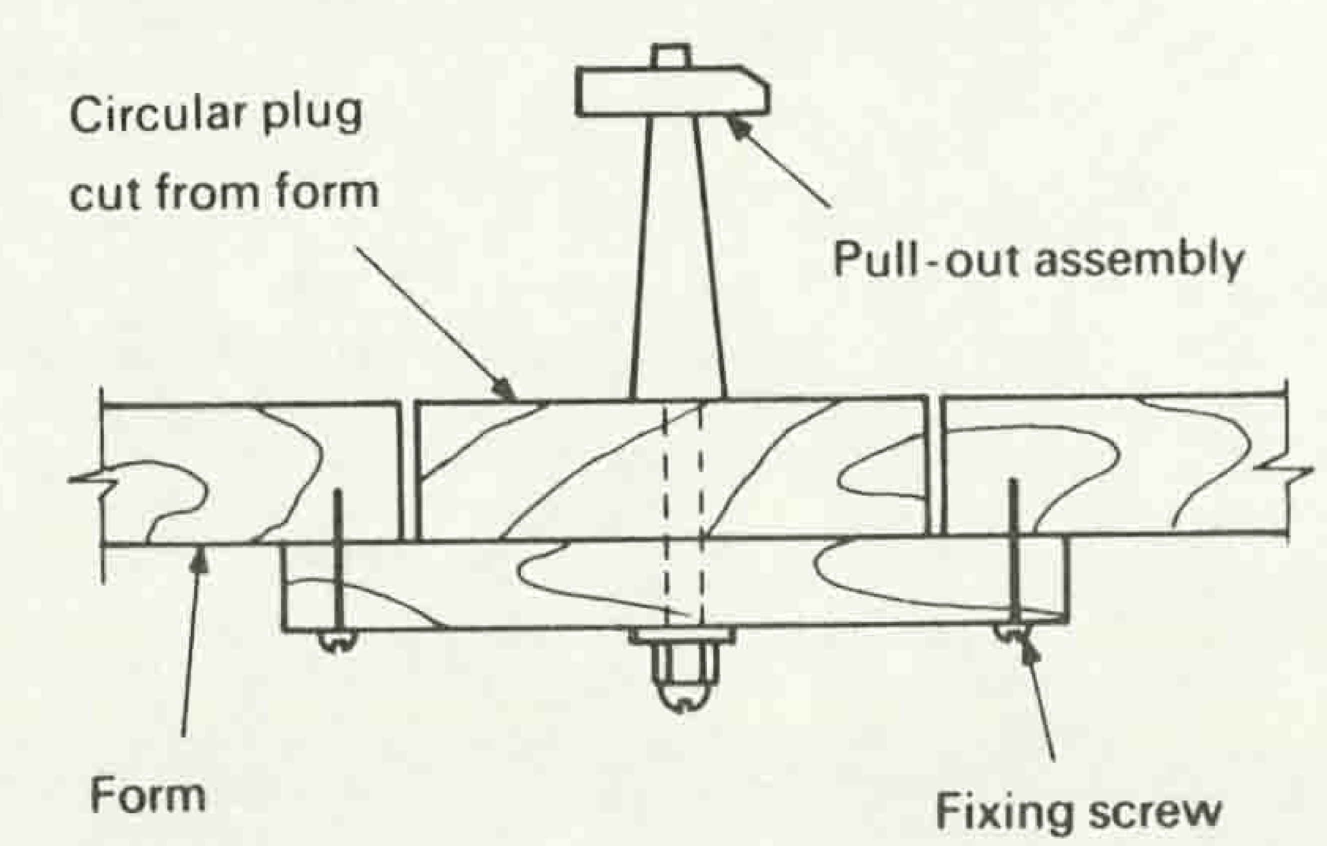

Figure 4.12 Arrangement for formwork stripping time tests.

determination method and may form the basis of specification compliance assessment. Its use in Scandinavia for in-situ strength monitoring has been considerable, and this is likely to spread in the future.

4.2.1.2 North American pull-out methods. In the early 1970's Richards (53) published data from tests made using equipment of his design, the basic form of which is shown in Figure 4.13. During subsequent years a number of test programmes were reported in the United States and Canada using this approach and other comparable test assemblies. These were sufficient to confirm the potential value of the method, and American (54) and Canadian standards have subsequently been developed. ASTM C900-78T allows considerable latitude in the details of the test assembly whilst specifying ranges of basic relative dimensions. It is intended that a hydraulic ram is used for load application, which should be at a uniform rate over a period of approximately two minutes. The depth of test may be greater than that of the Lok-test (section 4.2.1.1) although this equipment does satisfy the requirements of both American and Canadian standards. Indeed, recent reports (52) suggest that use of the commercially available Lok-test system is increasing in these countries in preference to other versions of the method.

The failure surface will be less precisely defined than with the Lok-test because of the range of allowable dimensions, but Richards (55) suggests that an apex angle of 67° is most satisfactory. Although little theoretical work has been published relating to this type of insert it is likely that mechanisms will occur which are similar to those for the Lok-test. Presentation of results, however, according to ASTM C900 (54) should be in the form of a pull-out strength (f_p) calculated from the ratio of pull-out force to the failure surface area

$$f_p = \frac{F}{A}$$

where F = force on ram
and A = failure surface area

A may be calculated from

$$A = \frac{\pi}{4}(d_3 + d_2)(4h^2 + (d_3 - d_2)^2)^{1/2}$$

where d_2 = diameter of pull-out insert head
d_3 = inside diameter of reaction ring
h = distance from insert head to the surface.

The published numerical data relating to this method is not extensive, but correlation coefficients of 0.99 are claimed (55) between pull-out and core strengths, with a relationship of pull-out strength = 0.21 × core strength. Coefficients of variation on testing would appear to be influenced by the

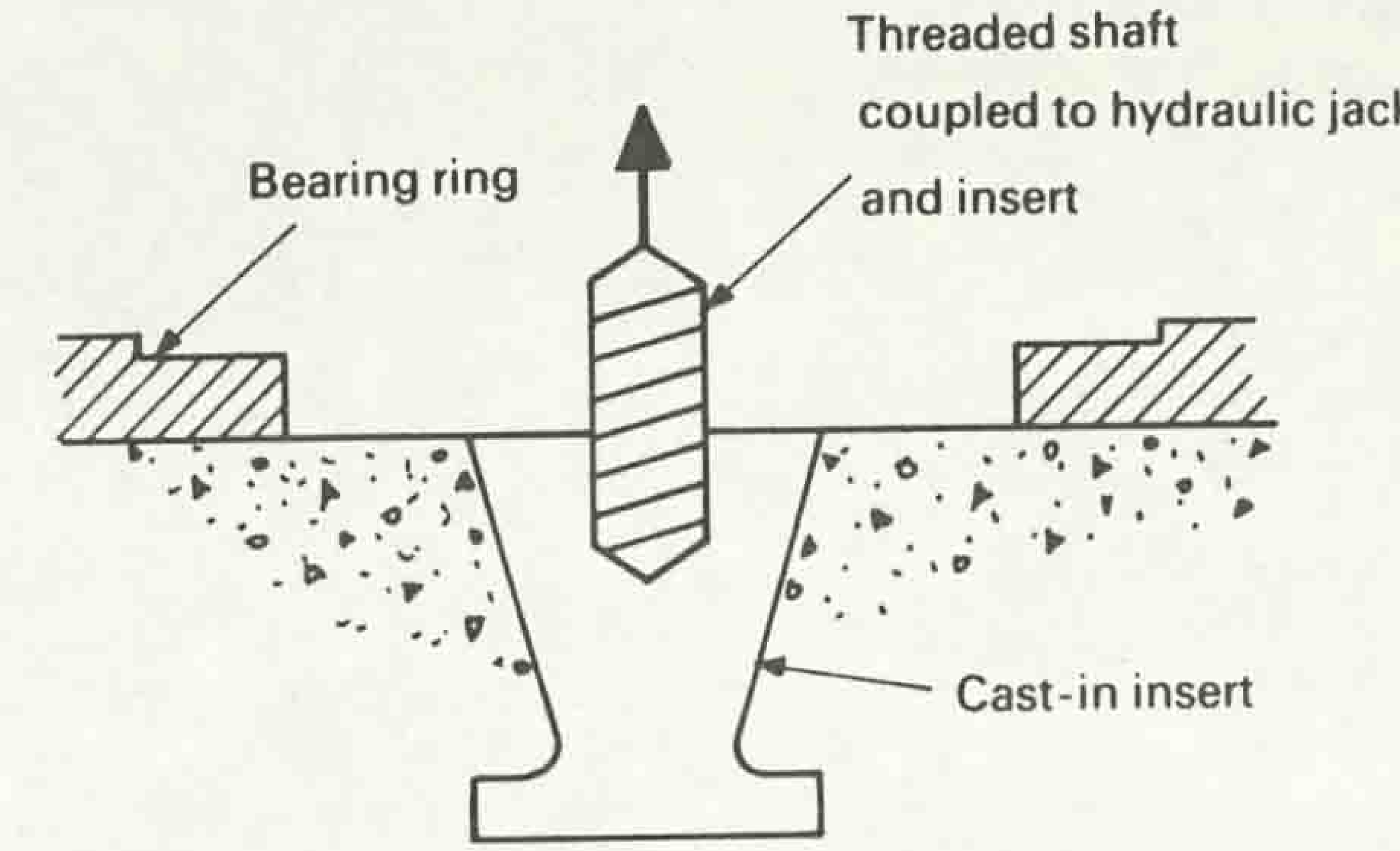

Figure 4.13 "American" insert.

nature of the insert, but Richards (55) also reports values as low as 7% for precision turned inserts.

Applications are obviously limited to preplanned situations and will be similar to those discussed for the Lok-test, whilst similar limitations will also apply.

4.2.2 *Drilled hole methods*

These offer the great advantage that use need not be preplanned. Early proposals from the USSR involved bolts grouted into the holes, but more recently two alternative methods have been developed and are both commanding interest. In 1977 the use of expanding wedge anchor bolts was proposed by Chabowski and Bryden-Smith (56), working for the Building Research Establishment. Their technique was initially developed for use with pretensioned high alumina cement concrete beams and is known as the internal fracture test. This work has subsequently been extended to Portland cement concretes (57), whilst the author has suggested that an alternative loading technique offers greater reliability (58). In Denmark, work on the Lok-test (see 4.2.1.1), has been extended (51) to produce the Capo test (Cut and Pull Out) in which an expanding ring is fixed into an underreamed groove, producing a similar pull-out device to that used for the Lok-test.

Recent research in Canada (59) has also considered drilled hole methods incorporating split sleeve assemblies, as well as reviving the concept of bolts set into hardened concrete using epoxy. This suggests that, despite practical problems and high test variability, both of these approaches are worthy of future development. There is little doubt that if a reliable drilled hole pull-out approach could be established, it would be extremely valuable for in-situ concrete strength assessment, especially when the concrete mix details are unknown.

4.2.2.1 Internal fracture tests. The basic procedures for this method (56) are as follows. A hole is drilled 30–35 mm deep into the concrete using a roto-hammer drill with a nominal 6 mm bit. The hole is then cleared of dust with an air blower and a 6 mm wedge anchor bolt with expanding sleeve is tapped lightly into the hole until the sleeve is 20 mm below the surface (Figure 4.14). Verticality of bolt alignment relative to the surface can be checked using a simple slotted template.

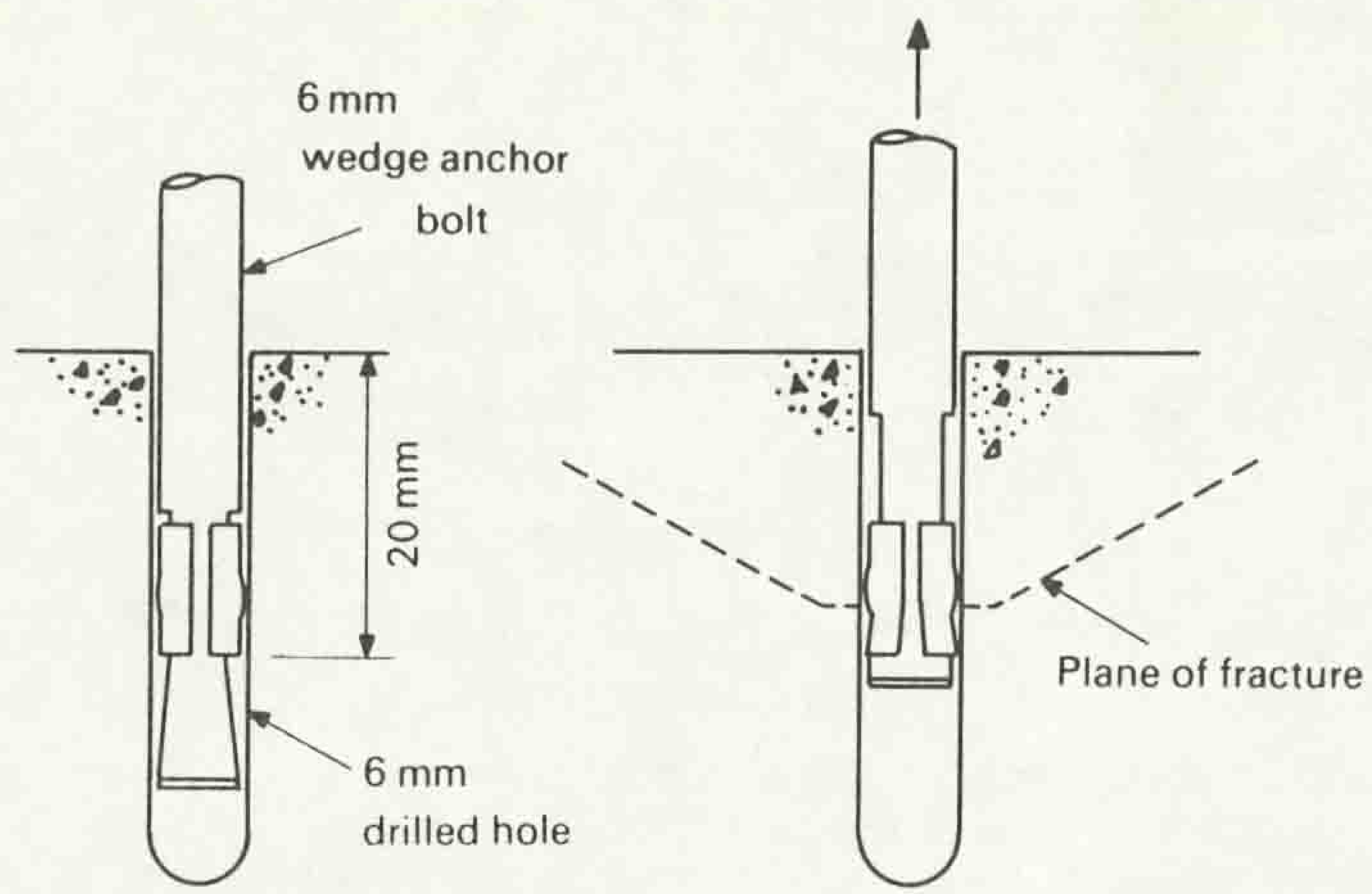

Figure 4.14 Internal fracture test.

The bolt is loaded at a standardized rate against a tripod reaction ring of 80 mm diameter with three feet, each 5 mm wide and 25 mm long. If necessary, shims may be used to correct for minor bolt misalignments. After applying an initial load to cause the sleeve to expand, the force required to produce failure by internal fracture of the concrete is measured. This will be the peak load indicated by the typical load/movement pattern in Figure 4.15. If the load is reduced once this peak has been reached there is likely to be no visible surface damage and it has been suggested that the bolt can be sawn off. If load application continues, a cone of concrete will be pulled from the surface, often intact, and considerable "making good" may be necessary. It has been found by the author (58) that the load application method greatly influences the value of pull-out force required. The rate of load application affects not only the magnitude but also the variability of the results, and continuous methods yield more consistent results than if pauses are involved. Whatever loading method is adopted, it is essential that any calibration curves which are used relate specifically to the procedures followed. The importance of this is illustrated by comparison of the curves for two specific methods shown in Figure 4.19.

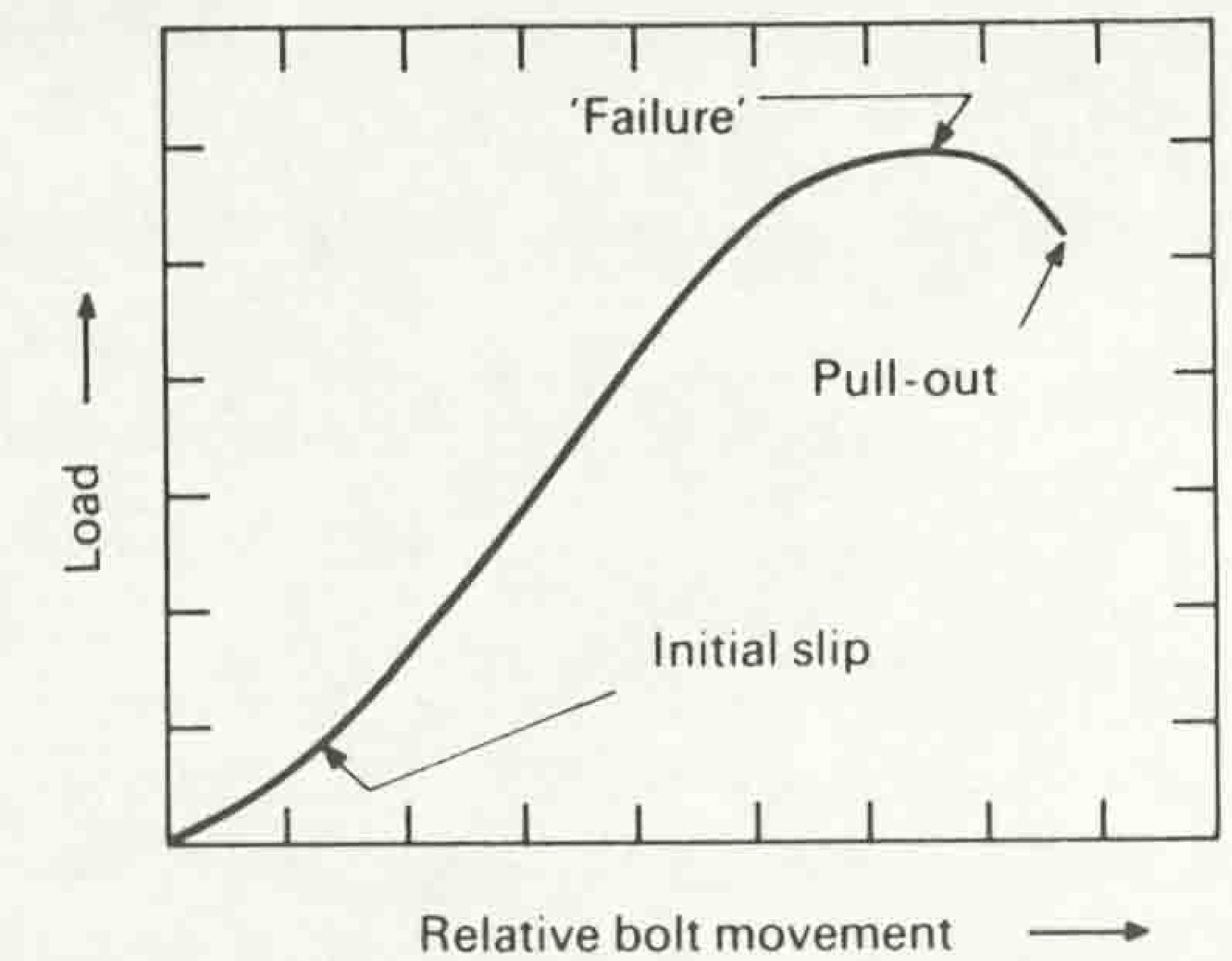

Figure 4.15 Typical loading curve.

In the *BRE loading method*, the BRE (56) recommend that load is applied through a nut on the greased bolt thread by means of a torquemeter, which is rotated one half turn in 10 seconds and released before reading, the procedure being repeated until a peak is passed. The tripod assembly (Figure 4.16) incorporates a ball race and a facility for automatic alignment with the axis of the anchor bolt to ensure that an axial load is applied with no bending effects. Early tests also required a load cell, but subsequently the method was

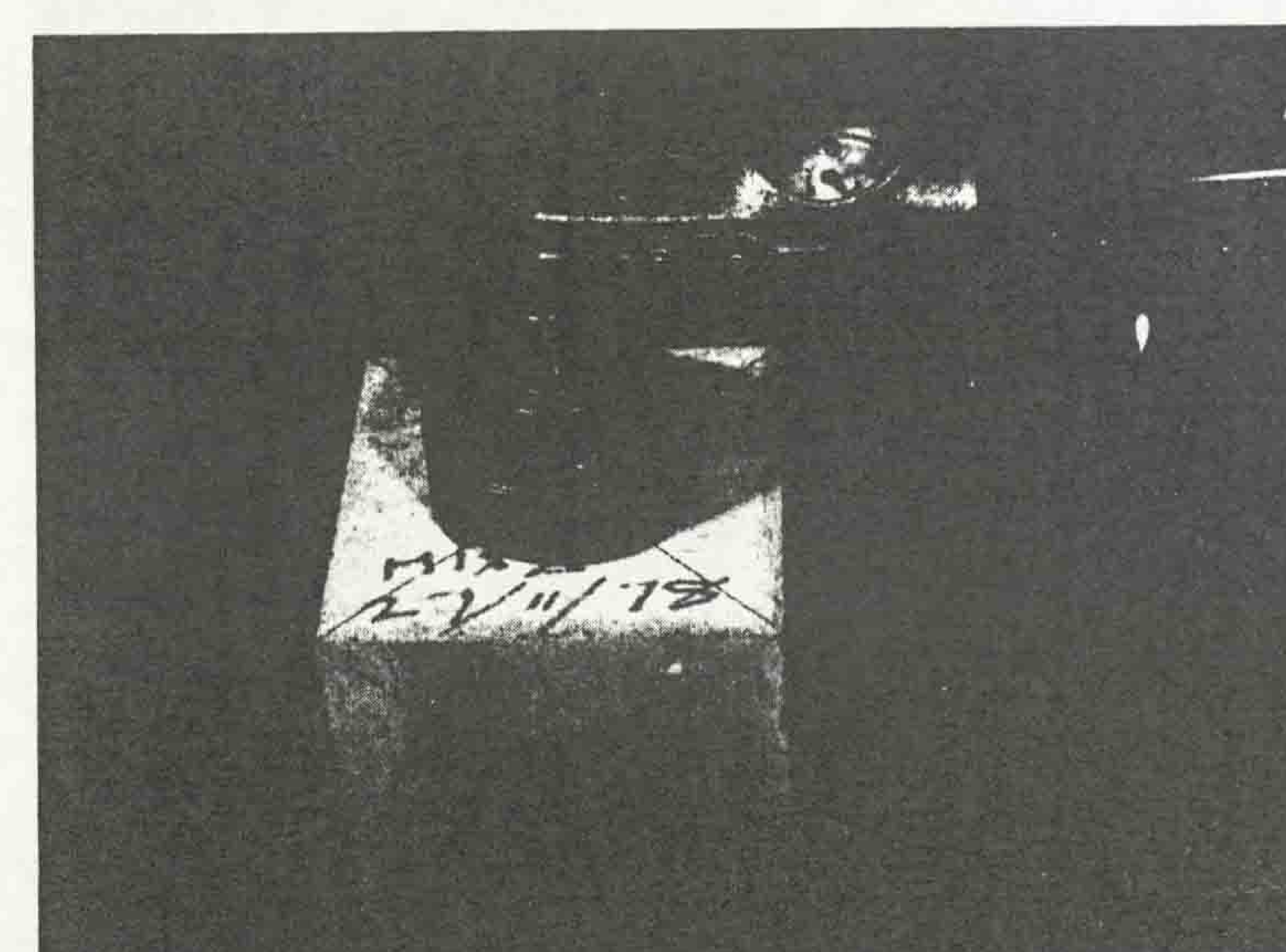

Figure 4.16 Torquemeter loading method.

developed on the basis of calibrations between measured torque and compressive strength.

Whilst this loading method is simple to use on both horizontal and vertical surfaces it suffers from two main disadvantages. Firstly, some torque is inevitably applied to the bolt, depending to some extent on the amount of grease on the thread, and this may reduce the failure load and increase the scatter obtained from individual results. Secondly, the torquemeter is relatively insensitive, and determination of the peak load is hindered by the use of settling pauses in the load procedure.

An alternative mechanical loading method has been developed by the author (58) which has the advantage of providing a direct pull free of twisting action. The latest version of this equipment is shown in Figure 4.17. The use of a proving ring for load measurement is sensitive, and provides a continuous rather than a settled reading, with the result that the variabilities due to load application and measurement are reduced. Loading is provided at a steady rate, without pauses, by rotating the loading handle at the rate of one

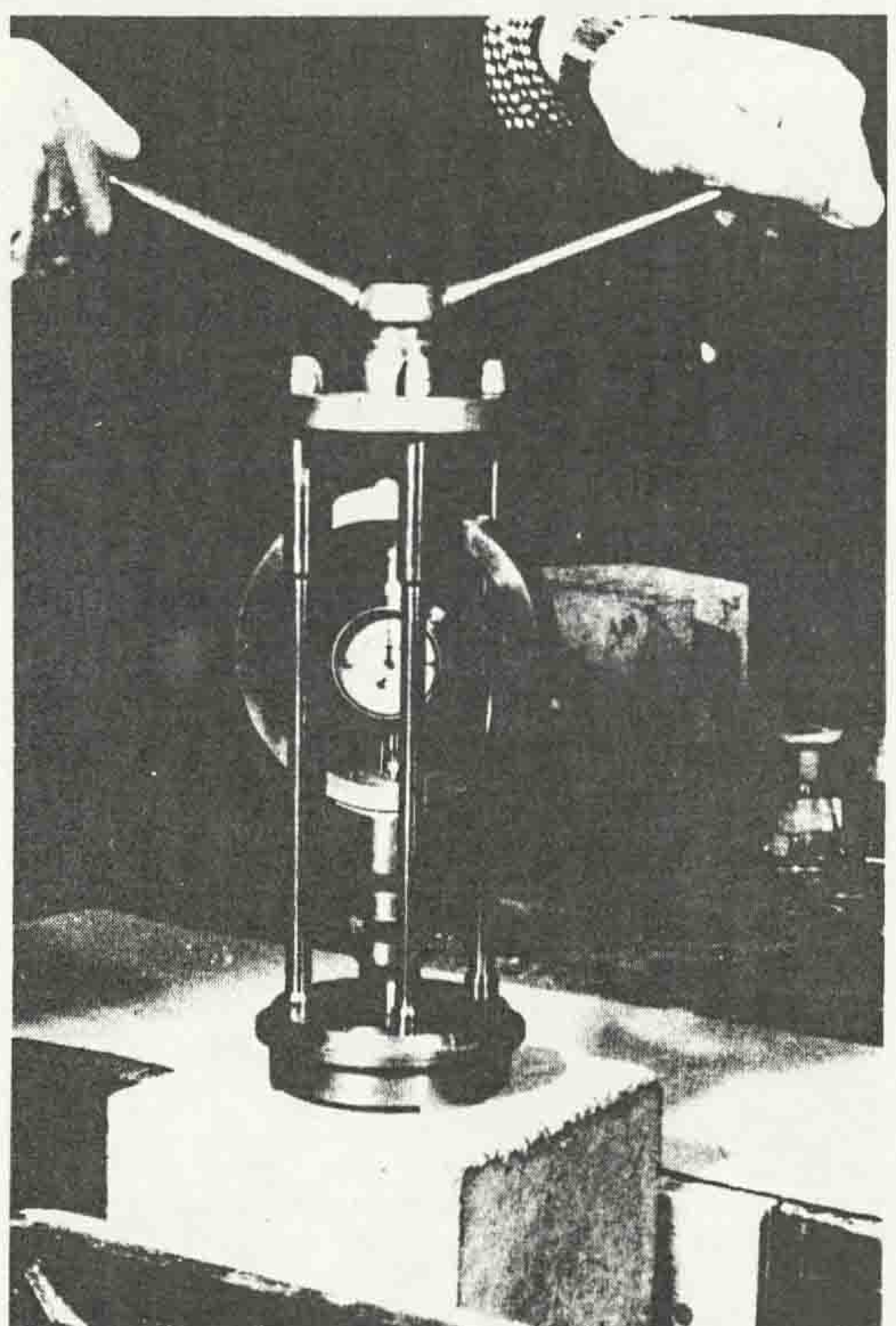

Figure 4.17 Proving ring loading method.

revolution every 20 seconds. Calibration charts have been produced for this loading procedure which relate compressive strength to direct force, and the variability due to testing using this approach has been shown to be lower than for the BRE method.

The load transfer mechanisms in these methods are complex, due to the concentrated localized actions of the expanding sleeve. The location of large aggregate particles relative to the sleeve will further complicate matters and affect the distribution of internal stresses. This is partially responsible for the high test variability found for the internal fracture test. The basic test assembly dimensions have been determined largely from practical considerations of suitable magnitude of force, and obtaining a depth of test to generally avoid surface carbonation effects whilst minimizing likely reinforcement interference. As the name of the method implies, failure is thought to be initiated by internal cracking. Attempts have been made to represent this theoretically (60) on the basis of an observed average failure depth of 17 mm which corresponds to a failure half angle of 78°. This is considerably greater than the likely angle of friction for the concrete of 37°, and application of the modified Coulomb failure criterion (as for the Lok-test, section 4.2.1.1) indicates failure by a combination of sliding and separation. This confirms the dependence of the pull-out force upon the tensile strength of the concrete. Paterson (61) has also discussed the general theory associated with the load capacity of fixings, but in practice the test method at its present stage of development relies upon empirical calibrations.

Tests on cubes which were subsequently crushed have been described for a variety of mixes by both Chabowski (57) and the author (58). Both reports indicate a reduction in crushing strength of 150 mm cubes of the order of 5% as a result of previous internal fracture tests on the cube. This must be taken into account when developing a calibration, unless undamaged specimens are available for comparison. There is also agreement that for practical purposes, mix characteristics (cement type, aggregate type, size and proportions) will not affect the pull-out/compressive strength relationship for natural aggregates. An upper limit of 20 mm on maximum aggregate size is suggested in view of the small test depth. The author has also shown that the variability of results increases with aggregate size, and that moisture condition and maturity have negligible effects. These features represent the chief advantage of this approach compared with other non-destructive or semi-destructive methods, although the scatter of results is high, as illustrated by Figure 4.18 where each point represents the average of six tests on a cube. This means that a considerable number of specimens are required to produce a calibration curve.

The effects of precompression, as may be experienced in columns or prestressed construction, have also been examined by Chabowski (57) who

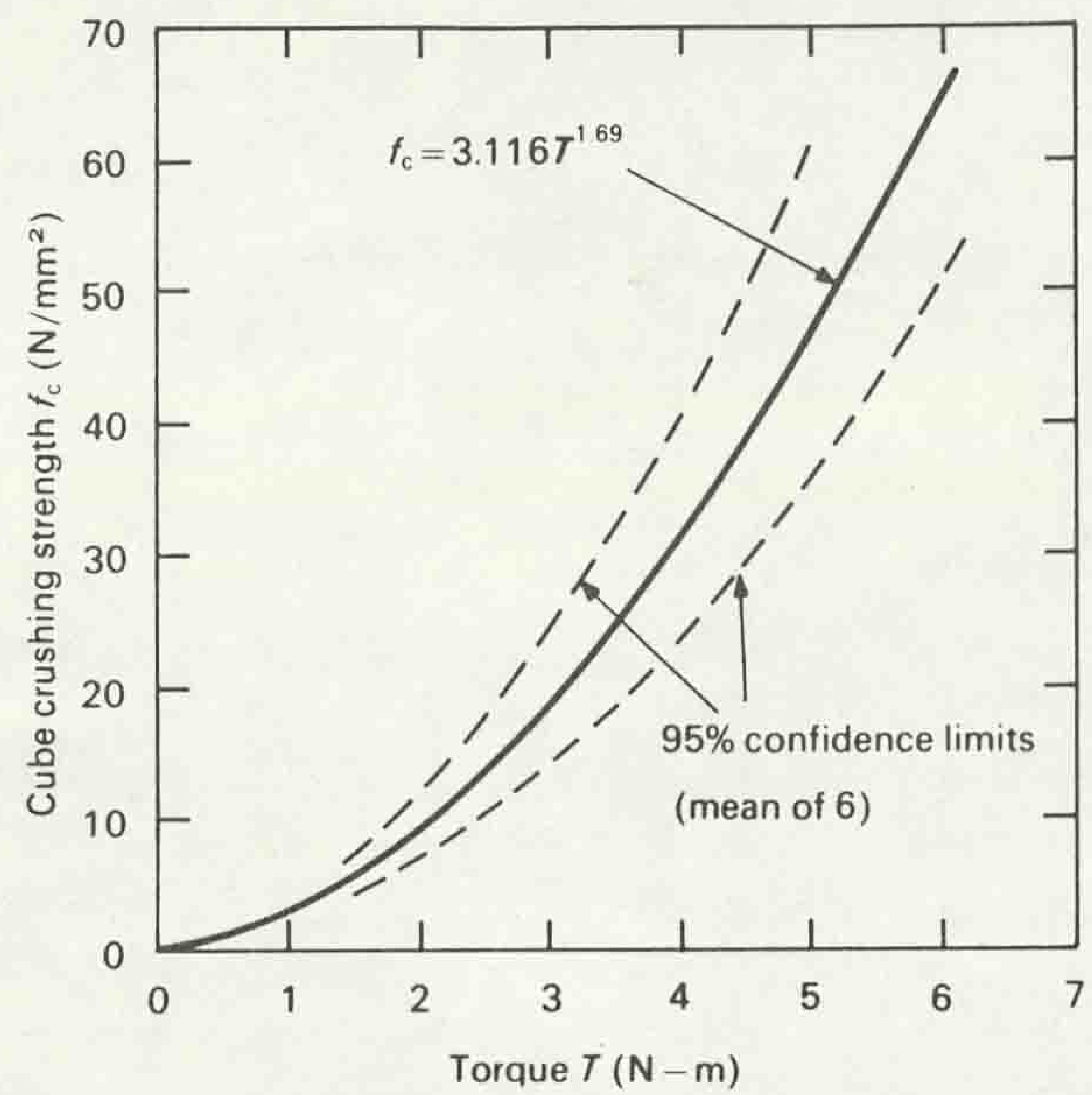

Figure 4.18 Typical compressive strength/torque calibration (based on ref. 57).

concludes that there is no clearly defined influence. Although a trend towards an increase in pull-out force with increasing compression is indicated, it is suggested that (provided zones of low stress are selected) this effect can be ignored in practice. The author (58) has reported tests on beams in flexure which demonstrate a similar conclusion, although the variability of results appears to increase with increasing stress. Tensile stress will have a similar effect, and tests must not be made adjacent to visible cracks. Surface carbonation is another effect which both investigators conclude can be neglected in most circumstances. Only in very old concrete where the depth of carbonation approaches the depth of the test will this effect have any influence. The shallow depth of test also offers the advantage that reinforcement is unlikely to affect results.

The influence of loading method has been indicated above. Results for the torquemeter loading method are generally expressed in the form of a compressive strength/torque relationship, but average force/torque ratio of 1.15 is reported by Chabowski (57). Comparative tests by the author (58) between the direct pull and torquemeter methods suggest a corresponding ratio of 1.4 which reflects the differences in loading technique. The average relationships for the two techniques are compared in Figure 4.19. It must also be pointed out that a calibration obtained by the author using the torquemeter suggests compressive strengths up to 20% lower than the BRE

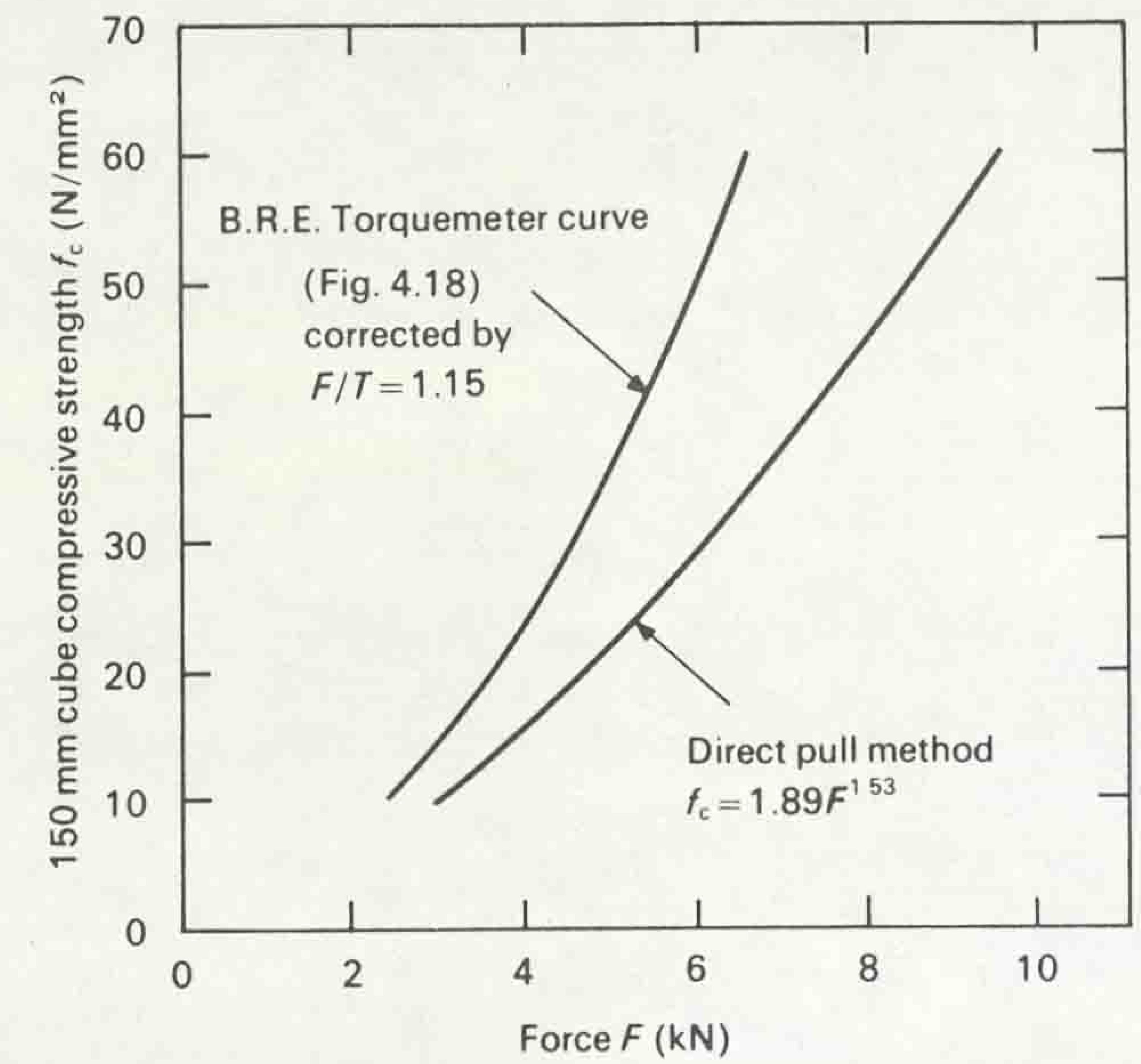

Figure 4.19 Comparison of calibration curves (based on refs 57 and 58).

calibration. A similar feature has also been indicated by Long (62), and cannot be ignored.

The variability of test results is high for a variety of reasons. These include the localized nature of the test, the imprecise load transfer mechanism and variations due to drilling. 95% confidence limits on estimated strength of ±28% based on the mean of six test results are claimed for the torquemeter load method (57), provided that individual results causing a coefficient of variation of greater than 16% are discarded. The author (58) has claimed a corresponding range of ±20% based on four results for 10 mm aggregates using the direct pull equipment. The average coefficients of variation observed for cubes of 20 mm maximum aggregate size were 16.5% for torque and 7.0% for direct force, with values 20% lower for 10 mm aggregate.

The chief advantage of the internal fracture test lies in the ability to use a general strength calibration curve for natural aggregates relating only to the loading method. Despite the variability, localized surface nature, and damage caused, this may be of particular value in situations where a strength estimate of in-situ concrete of unknown age or composition is required. This is especially true for slender members with only one exposed surface where cores or other direct techniques are not possible. The accuracy of strength estimate will be similar to that obtained by small cores but with considerable savings of time, expense and disruption.

4.2.2.2 The Capo test. This has recently been developed in Denmark (51) as an equivalent to the Lok-test for use in situations where use cannot be preplanned. The basic geometry of the Lok-test described in section 4.2.1.1 has been maintained, although the pull-out insert consists of an expanding ring inserted into an undercut groove. The name is based on the expression "cut and pull-out", and the procedure consists of drilling a 45 mm deep, 18 mm diameter hole, after which a 25 mm groove is cut at a depth of 25 mm using a portable milling machine. The expanding ring insert is then placed and expanded in the groove, and conventional Lok-test pulling equipment can be used as described previously. Testing may stop when a surface crack appears, or may continue to pull out the plug of concrete, in which case the ring may be recovered, recompressed and re-used up to three or four times.

Extensive laboratory testing programmes (51) have shown that the behaviour of this test is effectively identical to the Lok-test and that the strength calibration and reliability may be regarded as the same. It is claimed that the entire testing operation, including drilling, may be completed in about five minutes, and the equipment is available in the form of a comprehensive kit. In Denmark this method has been accepted as equivalent to the Lok-test and has been used on a number of projects for in-situ strength determination in critical zones. The potential areas of application are wide and although surface zone effects must be considered, the approach appears to offer the most reliable available indication of in-situ strength apart from cores. Although equipment costs are high, the damage, time and cost of testing will be considerably less than for cores. Problems may arise from the presence of reinforcement within the test zone, and bars must be avoided, but the value of this test is considerable in situations where mix details are not known.

4.3 Pull-off and break-off methods

These two approaches have been recently proposed to measure the in-situ tensile strength of concrete. Pull-off tests apply a direct tensile force to the concrete, whilst break-off test measurements are made in flexure. In both cases the fracture surface will be below the concrete surface, but will leave some surface damage that must be made good.

4.3.1 *Pull-off methods*

Pull-off tests have been described (63) which were developed initially in the early 1970's for suspect high alumina concrete beams. A steel probe is glued to the concrete surface by an epoxy resin and jacked off to measure the force necessary to pull a piece of concrete away from the surface. Two forms of

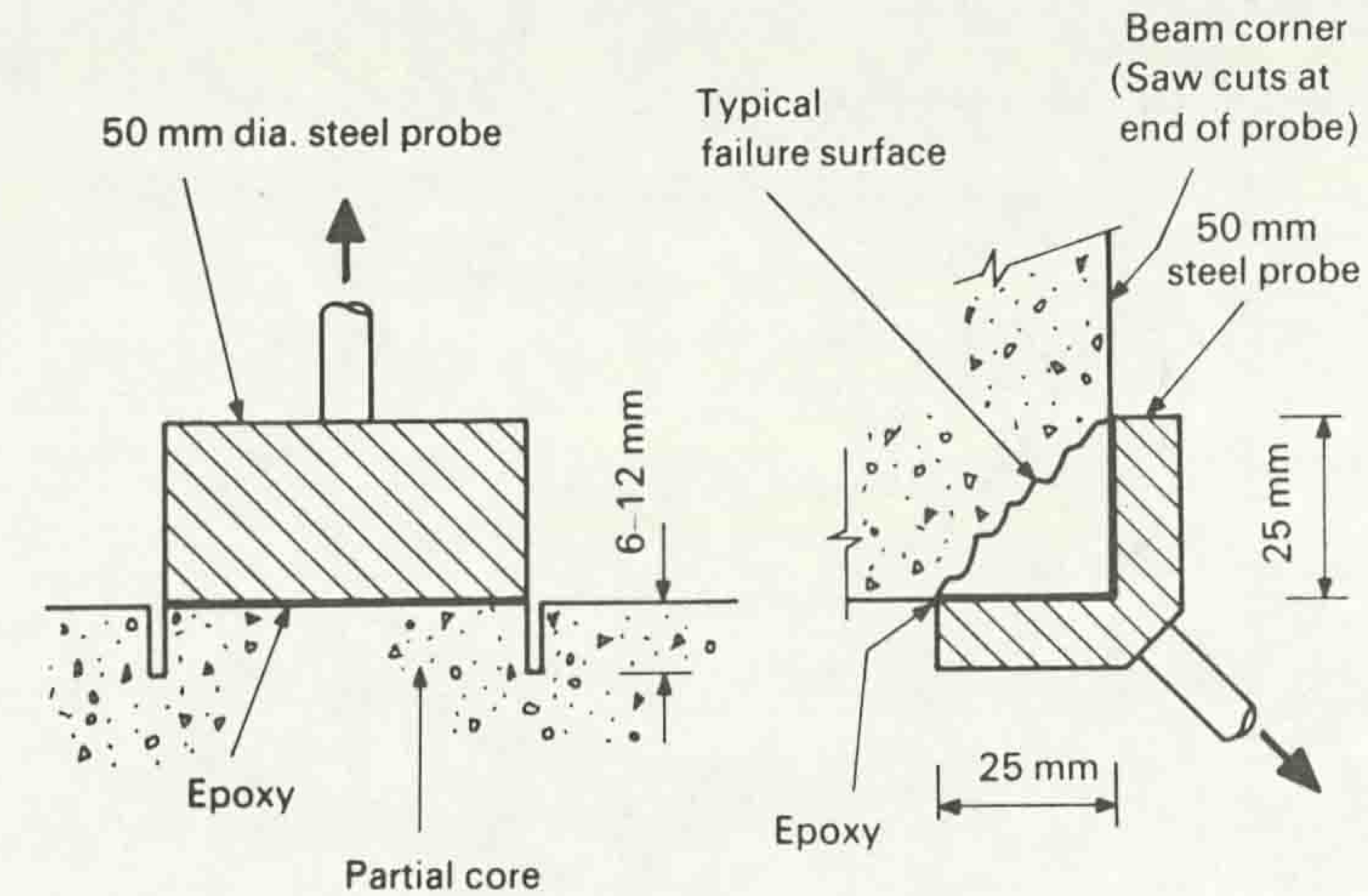

Figure 4.20 Pull-off probes.

probe have been used, one of which is suitable for a partially cored surface and the other for attachment to a beam corner, as illustrated in Figure 4.20.

Considerable care is needed in surface preparation, with probe surfaces etched by phosphoric acid and the concrete smoothed by emery cloth. A nominal tensile strength is calculated on the basis of the core area or throat area measured at right angles to the direction of pull. A good correlation is claimed by Long and Glass (62) between measured values and compressive strengths for a recent laboratory programme using 150 mm Portland cement cubes. A test coefficient of variation of 7.9% is reported with a range of the ratio of predicted/actual strength between 0.85–1.25 using the calibration developed.

The approach has not yet been developed outside the laboratory, although the reported results are promising and site procedures may follow. The influence of aggregate, cement and curing need to be fully investigated before reliable calibration curves can be produced for general use. In the form proposed, the failure surface is close to the concrete surface, and the influence of surface zone effects is uncertain. The method is particularly suitable for small section members, provided the surface damage is acceptable. It has been suggested that a long-term monitoring procedure could also be developed involving proof load tests at intervals on a series of permanent probes.

4.3.2 *Break-off methods*

Johansen (64) has recently reported the use of a break-off technique developed in Norway. This is intended primarily as a quality control test for concrete pavements, and makes a direct determination of flexural strength in a plane parallel and at a certain distance from the concrete surface. A tubular disposable form is inserted into the fresh concrete, or alternatively a shaped hole can be drilled, to form a slot of the type shown in Figure 4.21. The core left after removal of the insert is broken off by a transverse force applied at the top surface as shown. This force is provided hydraulically using specially developed portable equipment available under the name "TNS-Tester".

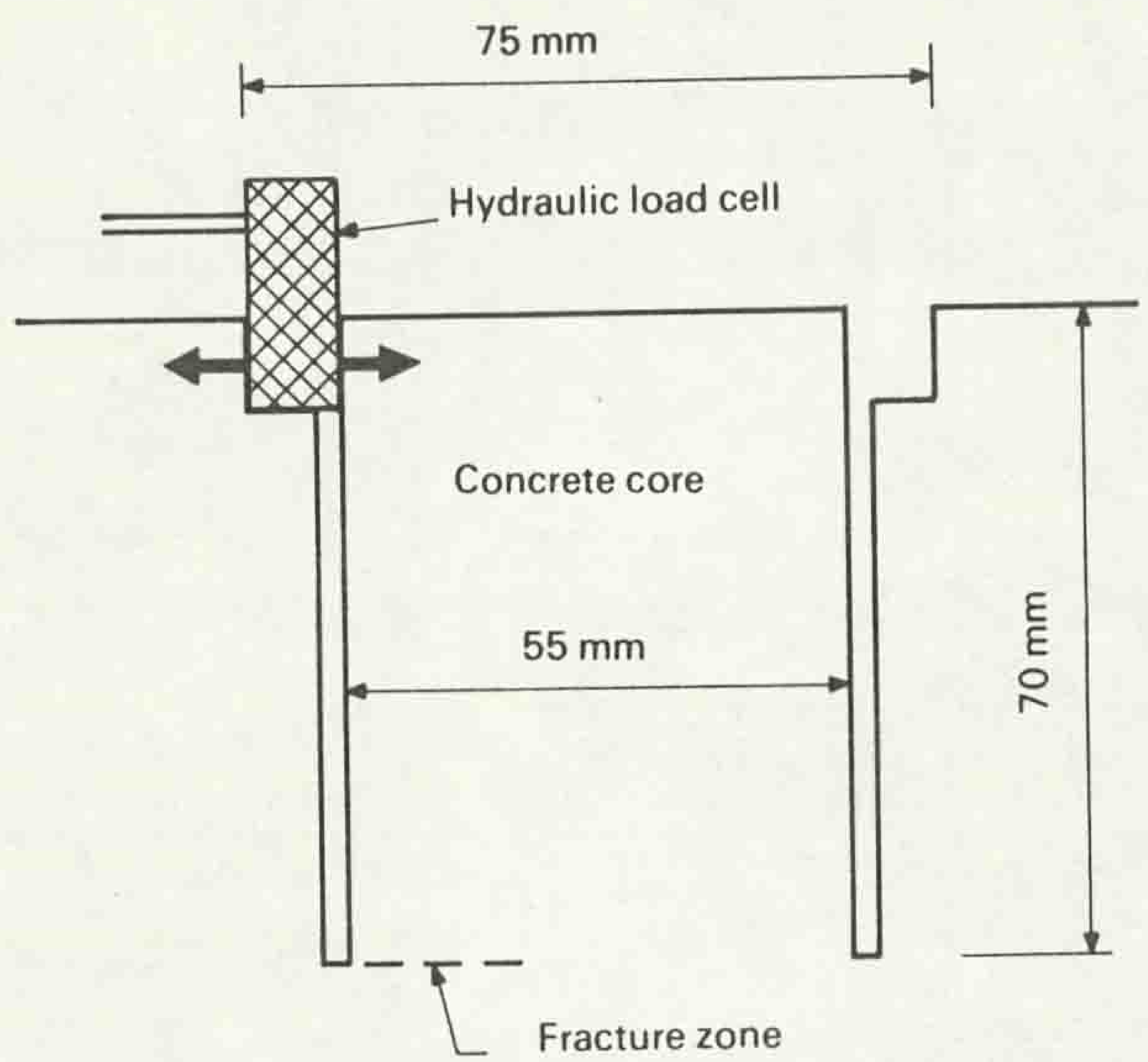

Figure 4.21 Break-off method.

The "break-off" strength calculated from the results has been shown to give a linear correlation with the modulus of rupture measured on prism specimens, although values were 30% higher on average. Values obtained (64) for an airfield pavement contract have suggested coefficients of variation of 6.4% for laboratory samples and 12.6% in-situ. It is suggested, however, that the mean of five test results should be used in view of high within-test variation.

It is claimed that the method is quick and uncomplicated, taking less than two minutes per test. Results are not significantly affected by the surface condition or local shrinkage and temperature effects. A correlation with compressive strength has also been developed which covers a wide range of concretes, but this is likely to be less reliable than a tensile strength correlation in view of the factors influencing the tensile/compressive strength relationship. The method is regarded as especially suitable for very young

concrete, and although leaving a sizeable damage zone, may gain acceptance as an in-situ quality control test where tensile strength is important. Although quicker than compression testing of cores, the use of results for strength estimation of old concrete may be unreliable unless a specific calibration relationship is available.

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Paper 12

"Testing by Penetration Resistance"

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Testing by penetration resistance

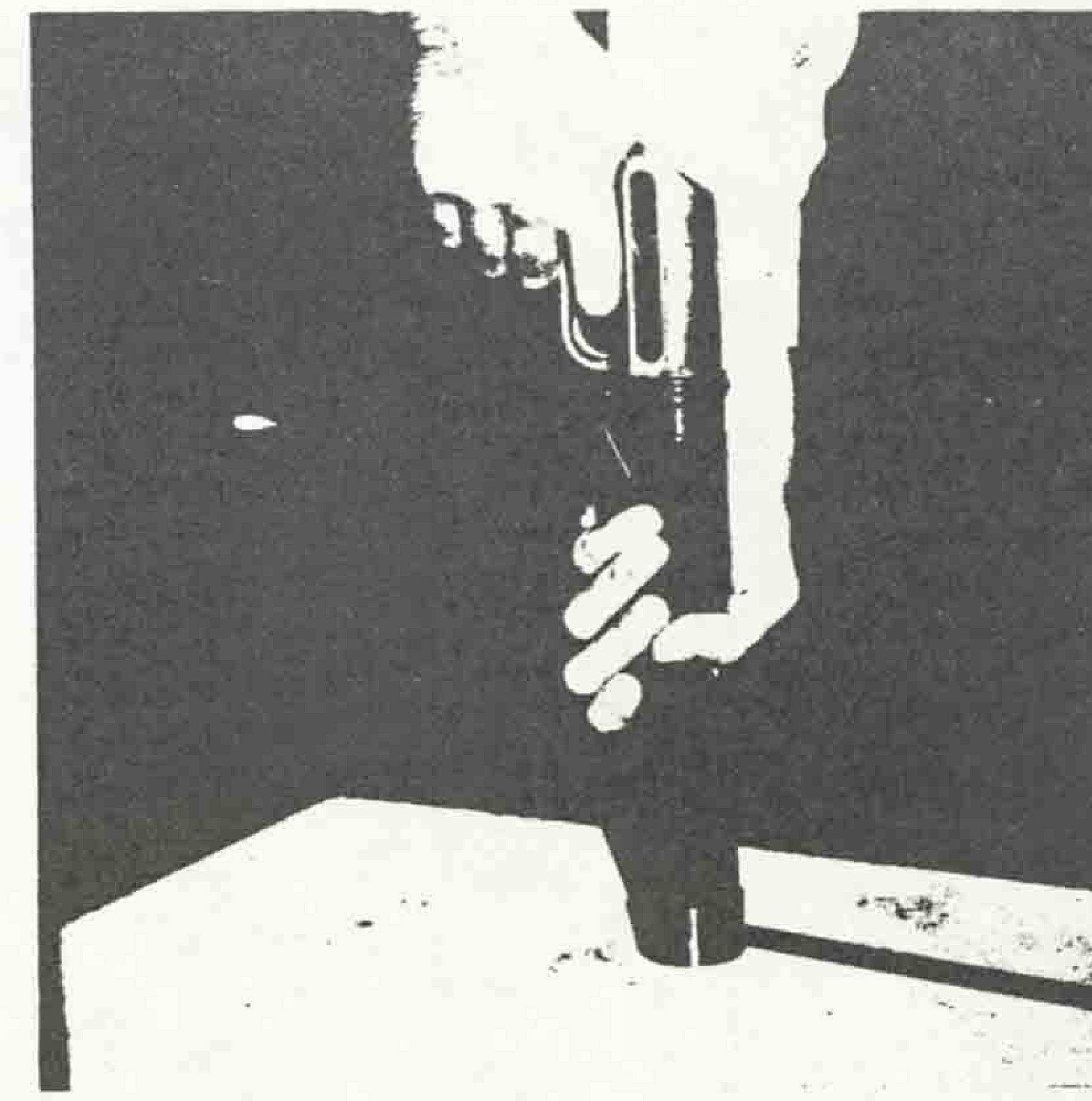


Figure 2: The driver during operation.

by J H Bungey

Following industrial training and experience with a consultant and Road Construction Unit, J H Bungey is currently lecturing in civil engineering specialising in structural concrete design, at the University of Liverpool. His research interests lie principally in the field of in situ assessment of concrete structures, and a number of papers have been published in this field.

ALTHOUGH the idea of measuring hardened concrete strength by firing a nail or bolt at the surface and measuring penetration is not new, it is only recently that a commercially-produced system has been available in the UK. This was developed in the USA during the 1960's and is known as the Windsor Probe Test[1]. The popularity of this method of testing has grown considerably in the USA and Canada, especially for monitoring strength development on site, and ASTM Standard C803-75T[2] deals with its use and application. Many authorities in those countries regard the test as an acceptable equivalent to site cores, and in some cases in lieu of control cylinders for compliance testing.

The test is essentially a form of hardness testing, and measurements only relate to the quality of concrete near the surface. Nevertheless it is claimed that it is a zone of between 25 and 75 mm below the surface which predominantly influences the penetration, in which case the test potentially offers many advantages over the popular Schmidt Rebound Hammer method. A theoretical relationship between the depth of penetration and concrete strength is difficult to evaluate, but consistent empirical relationships can be found that are virtually unaffected by operator technique. Although 'general' strength calibration tables are provided by the manufacturer there is evidence to suggest that, as with most other non-destructive tests, aggregate characteristics and other factors may have a considerable influence on results. Additionally, it is known that those calibration tables were based primarily on crushed rock aggregates typically used in the USA, thus it was considered worthwhile to commence a programme of tests on a range of British aggregates. These tests, undertaken by the author in the Department of Civil Engineering at the University of Liverpool, are continuing and will cover a wide range of aggregate characteristics, mix proportions and ages. The results presented here relate to the initial tests on gravel aggregates and demonstrate a number of important features relating to the use, calibration and application of this form of testing.

Test equipment and procedure

The bolt, or probe, which is fired into the concrete (Figure 1) is of hardened steel alloy with a blunt conical end to punch through surface aggregate particles and a shoulder to improve adhesion and ensure a firm embedment. A steel driving cap is screwed onto the threaded end whilst the plastic guide locates the probe in the barrel of the driver from which it is fired using a carefully standardised powder cartridge. This is designed to produce a probe velocity which does not vary by more than $\pm 1\%$, irrespective of orientation. For normal concretes a probe of 6.35 mm diameter and 79.5 mm length is used, although a larger diameter is available for lightweight concrete.

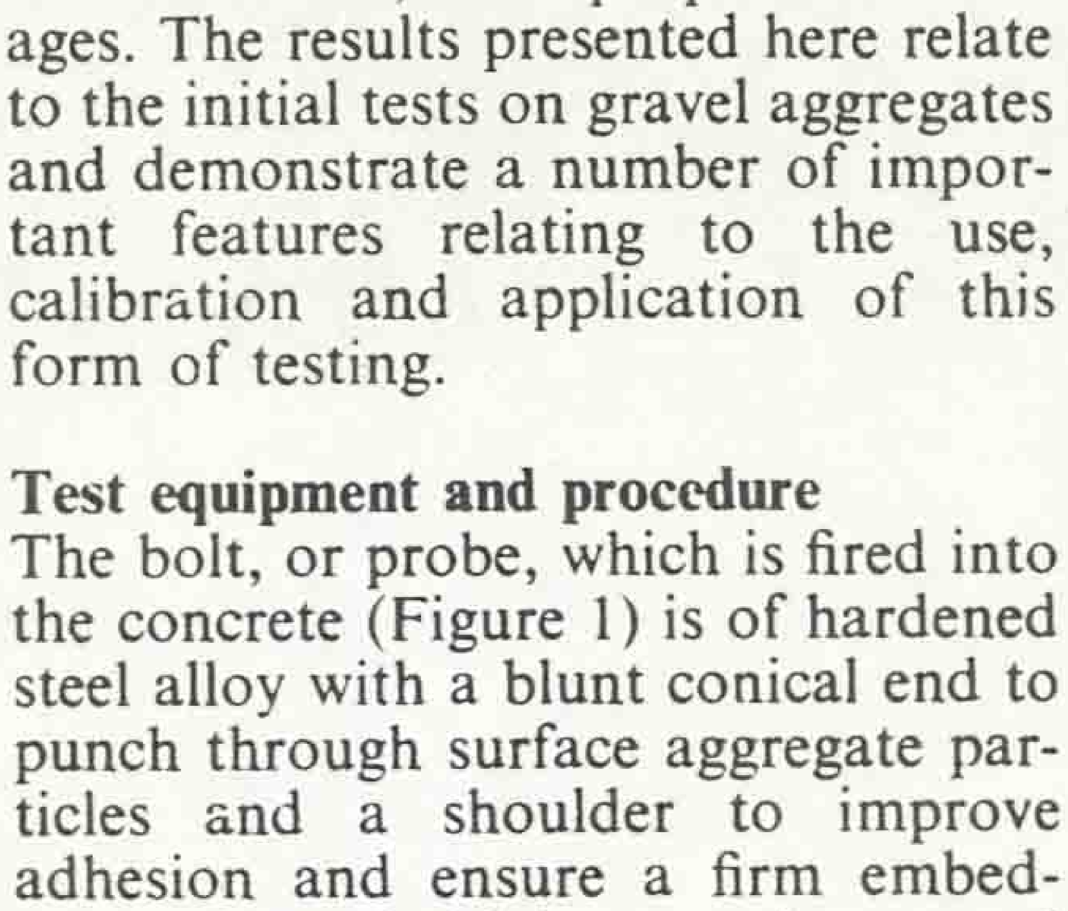


Figure 1: The hardened steel probe, which is fired into the concrete.

The driver, which is shown in operation (Figure 2), is pressed against a steel locating plate held on the concrete surface for firing, and after firing the exposed length of probe is measured with the aid of a graduated depth gauge. Although the verticality of the probe is important the effect of operator technique is small. Nevertheless, individual probes may be affected by localised influences such as particularly strong aggregate particles near the surface, and groups of at least three tests must be made and the results averaged. To cover the full range from low to high strength concrete it is necessary to use two different power settings of the equipment. 'Low Power' is used for concretes below about 26N/mm² cube strength, whilst 'Standard Power' is necessary for concretes above this strength to ensure firm embedment of the probe. There is no direct relationship between results for these two power levels, and calibration is thus made more complicated.

The power involved is considerable, and to avoid splitting of the member under test, minimum edge distances of 75 mm for low power and 100 mm for standard power are recommended. It has been found, however, in the course of this investigation that these values may not always be sufficient for an unreinforced block. Member thickness is also important and must never be less than twice the anticipated penetration, whilst it is recommended that the minimum probe spacing is 175 mm to prevent overlap of zones of influence.

It is known that aggregate hardness is a major factor in relating penetration resistance to strength[1]. Unless calibration charts are available for the particular aggregate source it is thus necessary to measure the coarse aggregate hardness by a scratch test using minerals of known hardness. This will yield a classification known as the Mohs number which may be used in conjunction with calibrations for that aggregate type.

Calibration tests

Whilst it may be possible to probe 150 mm cubes at low power, it is basically necessary to use larger slabs or beams for probing in conjunction with standard cubes for compression testing. The width of such blocks must be at least 200 mm and they must be at least 150 mm thick and of sufficient length to accommodate at least three probes whilst satisfying edge distance and spacing requirements. A major problem exists in ensuring similar compaction between the blocks and the cubes, and in this investigation ultrasonic pulse velocity measurements were used to compare specimens at the time of test.

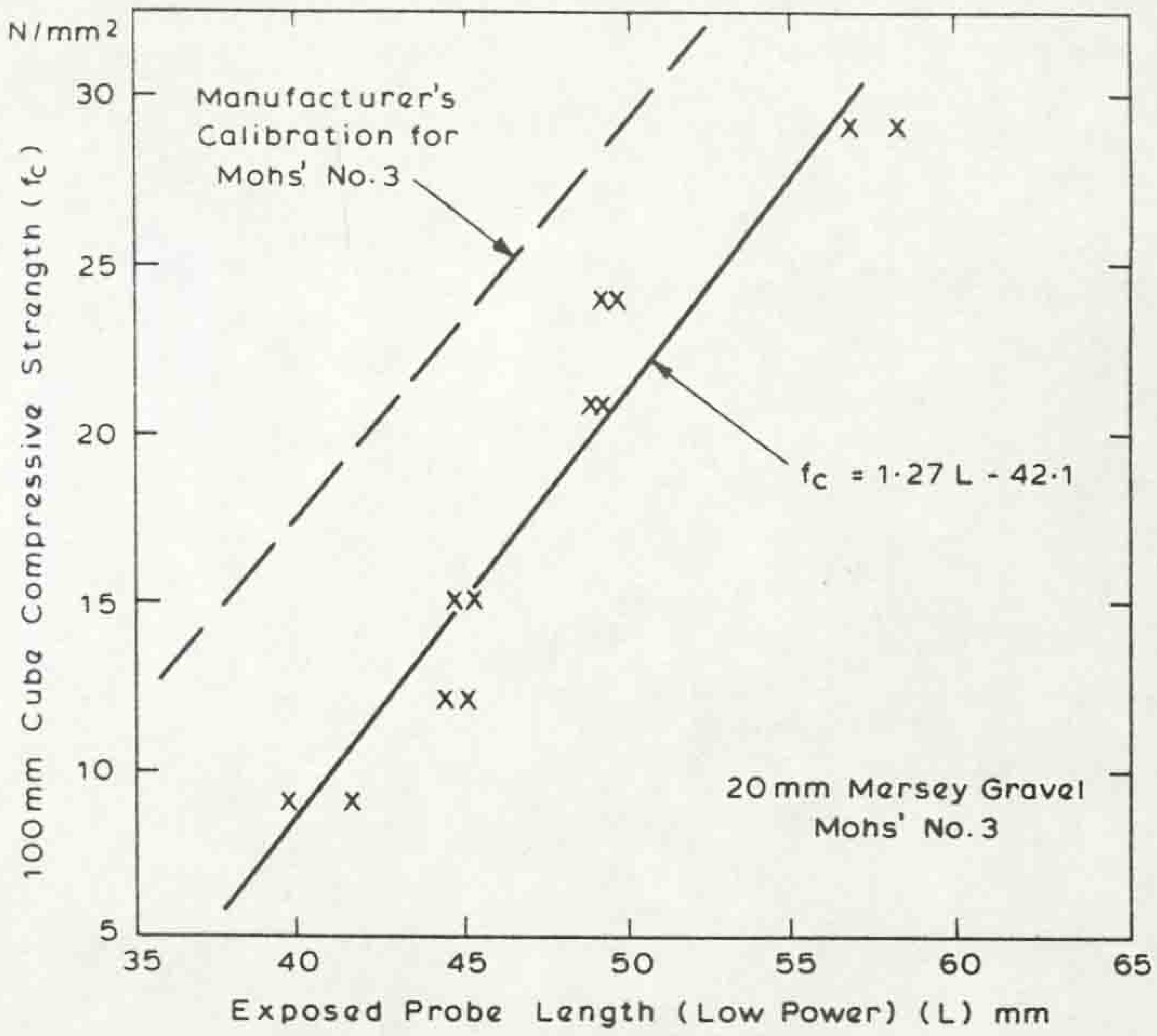
Beams of 1000 x 250 x 150 mm were cast in pairs and cured along with 100 mm cubes from the same batch. Strength variations were obtained by testing at varying ages when, after ultrasonic testing of cubes and beams, three cubes were crushed and at least two groups of three probes driven. An age-related pulse velocity calibration chart was produced for the mix at appropriate moisture conditions directly from the tests on cubes, and this was used to estimate the compressive strength in the beam at the location of each set of probes. In most cases it was observed from the UPV readings that the beam concrete was between 10% and 20% lower in strength than the cubes.

Experimental results

Tests have been carried out on concrete made with two widely differing 20 mm gravels. These were a hard North Notts gravel (Mohs No 7) and a soft Mersey gravel (Mohs No 3). Attention has been concentrated initially on tests at the Low Power setting and the results of these are shown in Figures 3 and 4, together with calibrations based on the tables provided by the manufacturer. For each aggregate type, two different mixes are involved with coarse aggregate/cement ratios between 4.0 and 6.0, and some tests were conducted on dry cured concrete, whilst others were on wet cured specimens tested whilst still saturated.

The influence of aggregate hardness is clearly apparent from Figure 5 and dominates over the other variables which have no discernible effect for the limited number of results available. The other significant feature of the results is the major discrepancy between both sets and the corresponding manufacturer's calibrations. Although the slopes are similar, the manufacturer's curves overestimate the true cube strength by up to 90% at the lower end of the range, and by between 35% to 50% at the upper end. The results available for standard power tests on mixes containing these aggregates are, at present, incomplete but show a similar trend, with the discrepancy remaining at between 8 and 10N/mm².

Figure 3: Experimental results for soft gravel.



The manufacturers indicate in their literature that for rounded gravels the crushing strengths of cubes or cylinders may be lower than suggested by probe results. Nevertheless, the extent of the discrepancy is disturbing. It is not possible to provide a detailed explanation at this stage since the mechanism of resistance to bolt penetration is not fully understood. It seems likely, however, that bond at the aggregate/matrix interface plays an important role.

Observations and conclusions

It is apparent that whilst a linear relationship exists between exposed probe length and compressive strength for specific mixes at ages up to 170 days, general calibration charts or tables are of little value and the importance of calibration for the particular aggregate type is emphasised. This conclusion has also been reached during a number of investigations in the USA[3] and obviously reduces the potential value of the test.

In comparison with the Schmidt Hammer, the method has the major advantage that the measurement relates to a greater depth within the concrete and is less dependent on surface condition (it is claimed[1] that finishes of less than 5 mm do not influence results). Furthermore, other variables such as age, aggregate content and moisture condition obviously have a far smaller influence.

On the other hand, the expense is considerably greater, since at current prices the equipment costs of the order of £900 with a recurrent cost of £3.50 per set of three probes. This, coupled with the greater surface damage which is caused by bolt removal, and the danger of splitting, means that penetration tests are unlikely to replace rebound tests except where the latter are accepted as being unsatisfactory.

A more likely usage of this test would seem to lie as an alternative to cores, where it offers advantages of ease of testing, instant results and cost. The accuracy of strength prediction is unlikely to match that of cores of 100 mm or over although the results suggest that 95% confidence limits of the order of ±20% may be possible given adequate calibration charts. This is comparable to small

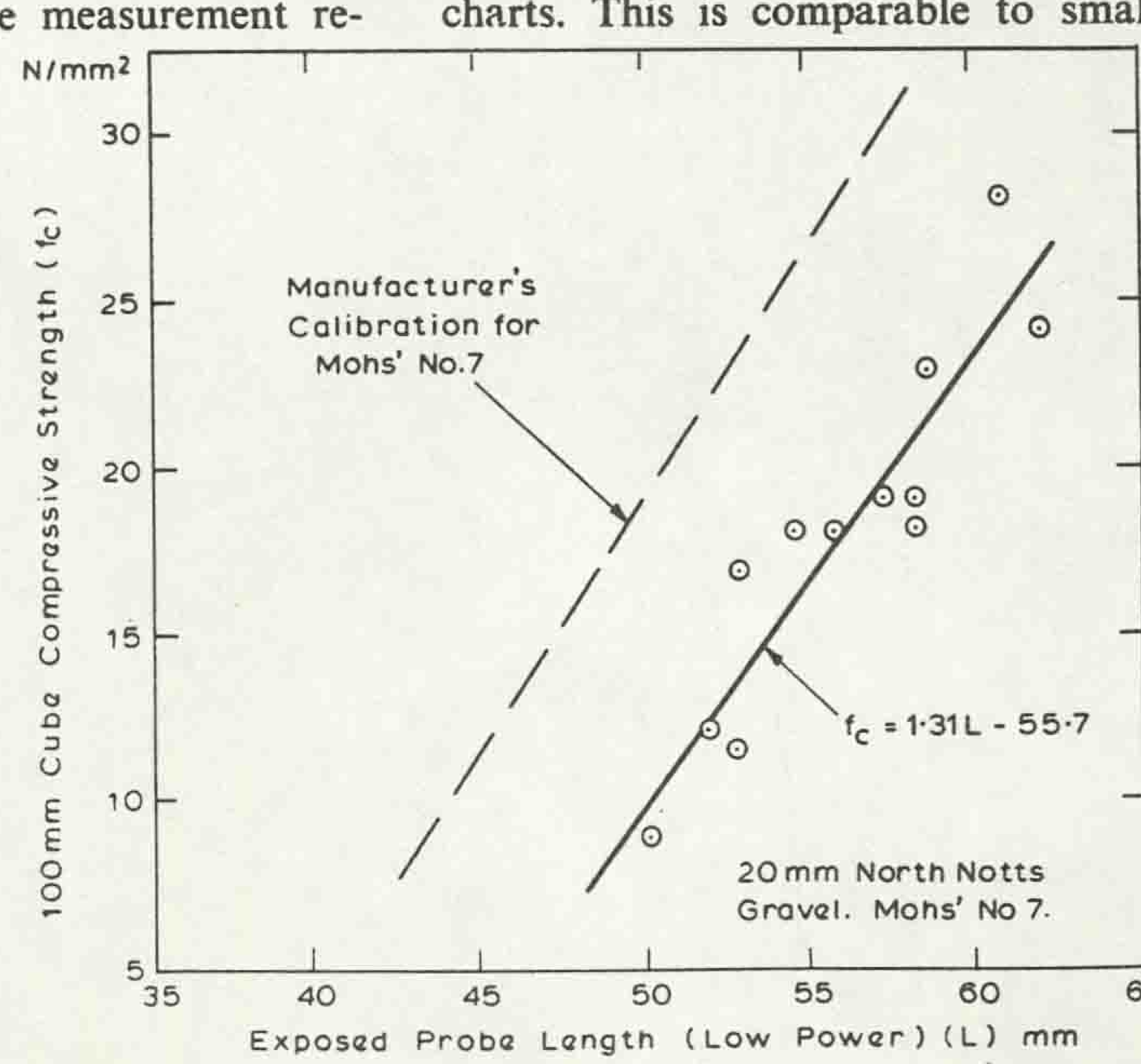


Figure 4: Experimental results for hard gravel.

diameter cores[4], and also to ultrasonics although in the latter case one exposed surface is not normally adequate and accurate calibration for site conditions may be considerably more difficult due to the influence of moisture conditions and maturity.

There is little doubt that the most reliable application of the penetration method lies in comparison of similar concrete. In the USA, where there is a trend towards in-place compliance testing, the equipment is used by a large number of organisations and many reported applications [3,5,6,7], include site checking for formwork or prop removal, post-tensioning, and determination of sub-standard areas or members. In these situations specific calibration charts can be produced and the advantages of speed and simplicity appear to outweigh the cost.

Whether or not the test will prove to be of significant value in the commonly-occurring situation of strength assessment of 'unknown' concrete remains to be seen, but it is clear from the results reported here that calibration charts based solely on aggregate hardness are inadequate and totally unreliable. A much more detailed study of the various parameters involved, coupled with an understanding of the actions taking place, is obviously necessary before the use of the method can be extended beyond the comparative situation with any degree of confidence.

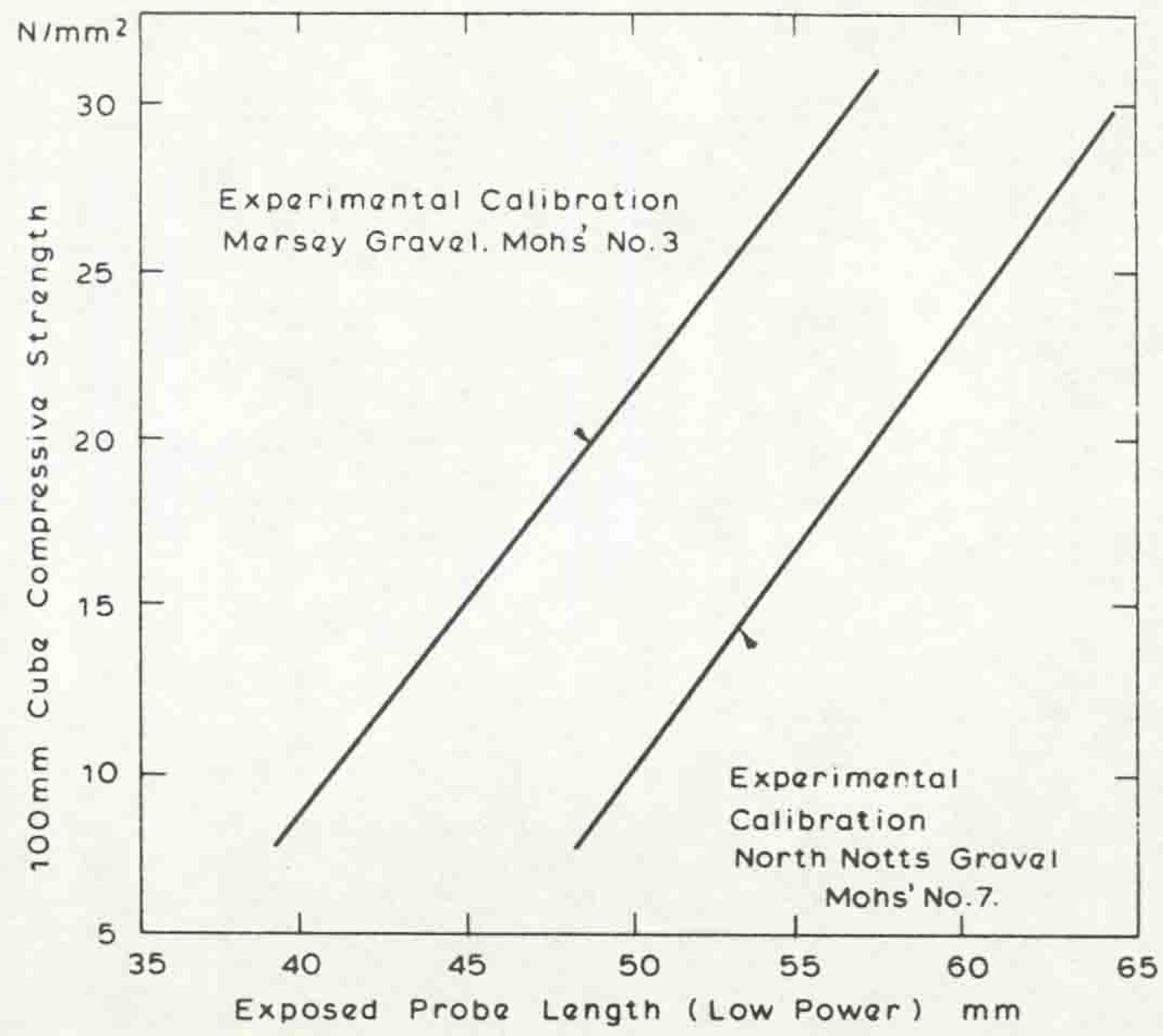


Figure 5: Comparison of 20 mm gravel calibrations.

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Paper 13

"An Appraisal of Pull-out Methods

of Testing Concrete"

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Engineering Technics Press

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AN APPRAISAL OF PULL-OUT METHODS OF TESTING CONCRETE

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Insitu strength assessment techniques which involve measurement of the force required to pull an insert from the surface of a body of concrete are examined and compared. Calibration procedures and strength correlations are presented, and the advantages, limitations and most appropriate applications are discussed.

1.0 INTRODUCTION

The concept of measuring the force needed to pull a bolt or some similar device from a concrete surface has been under examination for many years. Reports were first published in the USA and USSR in the late 1930's describing tests using cast-in bolts, but it was not until 30 years later that practically feasible tests of this type were developed in Denmark and the USA. Other techniques involving a drilled hole offer the great advantage that use need not be preplanned. Early proposals from the USSR involved bolts grouted into the holes, but more recently two methods have been developed in Denmark and the UK utilising mechanical load transfer.

Recent research in Canada (1) has also considered drilled hole methods incorporating split sleeve assemblies, as well as reviving the concept of bolts set into hardened concrete using epoxy. This suggests that, despite practical problems and high test variability, both of these approaches are worthy of future development. There is little doubt that if a reliable drilled hole pull-out approach could be established, it would be extremely valuable for in-situ concrete strength assessment, especially when the concrete mix details are unknown.

Three particular techniques have been examined by the Author in the course of recent laboratory studies and are examined in detail elsewhere(2). These involve a cast-in disc (Lok-Test), drilling and under-reaming followed by expansion of a steel ring to provide a similar configuration (Capo-Test), and the use of an expanding wedge anchor bolt in a drilled hole (Internal Fracture Test). The methods are currently at varying stages of incorporation into 'Standards' relating to concrete testing in North America and Europe, including the UK.

2.0 CLASSIFICATION

Results will relate to the surface zone only, but the approach offers the advantage of providing a more direct measure of strength and at a greater depth than surface hardness testing by rebound methods, and still requires only one exposed surface. The techniques must, therefore, be regarded as 'near to surface' test methods. A limited amount of physical damage will also be caused at, or just below, the surface of the concrete. Thus, whilst being 'non-destructive' in relation to the body of the concrete member under investigation, the methods must be classified as 'Partially

Destructive'.

3.0 TEST METHODS

3.1 'Cast-in' Methods

Two basic systems have emerged, both of which require a threaded insert which is placed prior to concreting. A bolt is then screwed into the insert and pulled hydraulically against a circular reaction ring. The principal difference between the two systems, developed in Denmark and Canada respectively, lies in the shape of insert and loading technique. In both cases a cone of concrete is "pulled-out" with the bolt, and the force required to achieve this can be translated to compressive strength by the use of an empirical calibration, or expressed as a 'pull-out' strength based on the ratio of pull-out force to the failure surface area. This type of test is covered by ASTM C900-78[3] which allows considerable latitude in the details of the test assembly and loading method whilst specifying ranges of basic relative dimensions.

The 'Lok-Test'[4], which was developed at the Danish Technical University in the late 1960's, has gained popularity in Scandinavia and is readily available in briefcase kit form. This method satisfies the requirements of ASTM C900 and is also becoming established in the USA.

The insert (Fig. 1) consists of a steel sleeve which is attached to a 25mm diameter, 8.5mm thick anchor plate located at a depth of 25mm below the concrete surface. The whole assembly is coated to prevent bonding to the concrete, and rotation of the plate is prevented by the "cut-off". The sleeve is normally screwed to the shuttering, or fixed to a plastic buoyancy cup where slabs are to be tested. This is later removed and replaced by a 7.2mm diameter rod which is screwed into the anchor plate and coupled to a tension jack. Load is applied to the bolt by means of a portable hand-operated hydraulic jack with a reaction ring of 55mm diameter (Fig. 2).

The loading equipment determines the force required to cause failure, and can cover concrete with cube strengths up to 69 N/mm^2. The load is measured with an accuracy of $\pm$ 2% over normal operating temperatures, and a precision valve system combined with a friction coupling ensures a constant loading rate of 30 $\pm$ 10kN/min. If load is released as soon as a peak is reached, only a fine circular crack will be left on the concrete surface, but if continued, a conical portion of concrete equal in diameter to the reaction ring will be pulled from the surface (Fig. 3).

The geometric configuration ensures that the failure surface is conical and at an angle of approximately 31^o to the line of pull. This is close to the angle of friction of concrete, which is generally assumed to be 37^o, and plasticity theory for concrete using a modified Coulomb's failure criterion indicates that where these are equal, the pull-out force is proportional to compressive strength. Finite element analyses of the failure mechanism have also indicated that failure is initiated by crushing rather than cracking of the concrete[5]. It is suggested that a narrow symmetrical band of compressive forces run between the cast-in disc and the reaction tube on the surface.

3.2 Drilled Hole Methods

These offer the greater flexibility of an insert fixed into a hole drilled into the hardened concrete.

(a) Capo-Test

In Denmark, work on the Lok-Test has been extended to produce the Capo test (Cut and Pull Out)[4] in which an expanded ring is fixed into an underreamed groove, producing a pull-out device with similar basic geometry to that used for the Lok-Test.

The procedure consists of drilling a 45mm deep, 18mm diameter hole with water lubrication. A 25mm diameter groove is then cut at a depth of 25mm using a portable milling machine (Fig. 4). An expanding ring insert is placed and expanded in the groove, and Lok-Test pulling equipment used as described previously. Testing must continue to pull out the plug of concrete, and the ring may be recovered, recompressed and re-used up to three or four times.

The equipment is available in the form of a comprehensive kit, and it is claimed that the entire operation may be completed in 5 minutes, although in the Author's experience 15-30 minutes, according to circumstances, would be more realistic. Reported laboratory programmes[4] indicate that the behaviour of this method is effectively identical to the Lok-Test and strength calibrations and reliability may be regarded as the same. Similar calibration procedures can be adopted, and although the number of tests by the Author is so far limited, indications confirm the published findings.

(b) Internal Fracture Test

In 1977 the use of expanding wedge anchor bolts was proposed by Chabowski and Bryden-Smith[5], working for the Building Research Establishment (B.R.E.). Their technique was initially developed for use with pretensioned high alumina cement concrete beams and is known as the internal fracture test. This work has subsequently been extended to Portland cement concretes[7], whilst the Author has suggested that an alternative loading technique offers greater reliability[8].

A hole is drilled 30-35mm deep into the concrete using a roto-hammer drill with a nominal 6mm bit. The hole is cleared of dust with an air blower and a 6mm wedge anchor bolt with expanding sleeve is placed into the hole (Fig. 5). Verticality of bolt alignment relative to the surface is checked using a simple slotted template.

The bolt is loaded at a standardized rate against a tripod reaction ring of 80mm diameter. After applying an initial load to cause the sleeve to expand, the force required to produce failure by internal fracture of the concrete is measured. Once the peak force has been reached, loading may be continued to pull-out the bolt together with a cone of concrete approximately 17mm deep, or the bolt may be sawn off and tapped below the surface leaving only a 6mm hole to be made good.

The loading method recommended by B.R.E. involves the use of a torquemeter to turn a nut on the greased thread of the bolt (Fig. 6). This is rotated one half turn in 10 seconds and released before reading, the procedure being repeated until a peak is passed. The tripod assembly incorporates a ball race and a facility for automatic alignment with the axis of the anchor bolt to ensure that an axial load is applied with no bending effects. Early tests also used a load cell, but subsequently the method was developed on the basis of calibrations between measured torque and compressive strength.

An alternative mechanical loading method has been developed by the Author which has the advantage of providing a direct pull free of twisting action. This is shown in Figure 7. Loading is provided at a steady rate, without pauses, by rotating the loading handle one revolution every 20 seconds. The equipment is sensitive, and provides a continuous rather than a settled reading, with the result that the variabilities due to load application and measurement are reduced.

The features of these alternative loading systems are compared in 4.2.

4.0 CALIBRATION

Calibration of measured values with concrete strength is obviously a crucial factor, and has been the subject of experimental studies by the Author.

4.1 Lok-Test

Published calibration data is available from the USA[9] and Denmark[4], and in most cases is based on tests on standard cylinders. In the investigation reported here it was decided to develop calibrations related directly to cubes as commonly used in the UK. It was not possible to use standard 150mm cubes in view of the recommended minimum edge distance requirement of 100mm, hence test specimens with dimensions 225 x 225 x 225mm were adopted. These were cast in timber moulds and used in conjunction with companion 100mm cubes for compressive strength testing.

Lok-Test inserts were cast into all six faces of the specimens, which were compared with the companion cubes by ultrasonic pulse velocity measurements prior to test. This was in order that allowance could be made for any minor differences in concrete quality between the specimens, which were all cast on a vibrating table and cured under identical conditions. A variety of mixes have been used incorporating 10mm and 20mm maximum size gravels of varying hardnesses, and crushed limestone. Mix proportions varied widely with tests performed at varying ages and moisture conditions. A total of over 150 measurements have been made, and the results are summarised in Figure 8, where each point represents the mean value and range of pull-out force for the six readings on a particular test specimen.

The relationship may be considered linear in form, and is relatively insensitive to aggregate type. The Manufacturers' recommended relationship:

$$\text{Lok Force} = (5 + 0.8f_{cyl})\ \text{kN}$$

can be approximately converted to an equivalent cube strength on the basis of

$$f_{cyl} = 0.8f_{cube} \qquad \text{to give}$$

$$\text{Lok Force} = (5 + 0.64f_{cube})\ \text{kN}$$

and this can be seen to be in remarkably close agreement with the measured values (Fig. 9).

The scatter of individual test values on a particular specimen was found to typically give a coefficient of variation of about 8-10%. It is estimated that the 95% confidence limits on a predicted compressive strength based on the mean of 6 Lok tests are about ± 20% using the combined calibration. For a particular aggregate type however, these can be reduced to about ± 10% when using a specific calibration for that aggregate as demonstrated by Figure 10.

4.2 Internal Fracture Test

Since a minimum edge distance of only 75mm is required tests can be performed on standard 150mm cubes, and both B.R.E. and the Author have found that if these are subsequently crushed they will yield values of strength approximately 5% lower than comparable sound specimens. Calibrations may thus be developed either by crushing cubes after internal fracture tests have been performed on all six faces, or by comparison with companion cubes. Following a large number of calibration tests the Author has confirmed that for practical purposes mix characteristics do not affect the calibration with compressive strength for natural aggregates, however there is consistent disagreement with the general curve of $f_c = 3.116\,T^{1.69}$ proposed by B.R.E. B.R.E.[7] for use with the torquemeter loading method.

Figure 11 compares the average curve obtained by the Author[8] with the B.R.E. proposals. It has been suggested that this discrepancy is due to an inadequacy of the bolt supporting collar in the tripod assembly, however the results of a recent series of tests utilising a modified component provided by B.R.E. do not confirm this. These results are superimposed on the curves of Figure 11. Calibration disagreement has also been noted by others[10].

The scatter of individual test results due to within test variability is high as a result of drilling imperfections, imprecise load transfer mechanism and the heterogenity of the concrete. The Author has frequently found a coefficient of variation in excess of 20% for tests on concrete made from 20mm maximum size aggregate using groups of six tests at a given location. The B.R.E.[7] have advocated a procedure whereby tests are continued until a group of 4 or 6 readings with a coefficient of variation of less than 16% is achieved. However, in the experience of the Author this offers few practical advantages in practice in comparison with a straightforward mean of 6 approach. The scatter in the relationship between the mean torque value at a location and the compressive strength obtained from cubes is considerable. 95% confidence limits on predicted strength of about ± 30% are likely but will only apply to a calibration relationship which has been developed by the user.

Whilst the torquemeter loading method is simple to use on both horizontal and vertical surfaces it suffers from two main disadvantages. Firstly, some torque is inevitably applied to the bolt, depending to some extent on the amount of grease on the thread, and this may reduce the failure load and increase the scatter obtained from individual results. Secondly, the torquemeter is relatively insensitive, and determination of the peak load is hindered by the use of settling pauses in the load procedure.

Calibration charts have also been produced for the Author's loading procedure which relate compressive strength to direct force[8]. The relationship obtained is significantly different from that derived from the torque calibrations afer correction to give an equivalent force (Fig. 12). This emphasises the importance of load application technique. Using this equipment the testing scatter is significantly reduced with typical values of coefficient of variation of 7% for 20mm aggregate and 5.5% for 10mm aggregate. The accuracy of strength prediction is improved and 95% confidence limits of ± 20% based on the mean of 4 values has been achieved for 10mm maximum size aggregate.

5.0 COMPARISON OF METHODS

It is evident that a considerable scatter of results is to be expected whichever technique is adopted, and it is thus necessary to perform at least 4 or preferably 6 individual tests to produce a mean value. The depth of test and dimensions of the insert are small in comparison to the size of individual aggregate particles, and this may be expected to be a contributory factor. It has been confirmed for the Internal Fracture Test that scatter reduces with maximum aggregate size, and it is to be expected that a similar feature will apply to the Lok Test and Capo Test.

Whilst the Internal Fracture Test is the simplest and cheapest of the methods and has the great advantage that it need not be preplanned before casting, the torquemeter loading approach clearly contributes significantly to a high variability of results. This can be overcome by the use of a more sensitive direct pull equipment. Whichever method is used to apply the load, it is essential that strength calibrations are prepared specifically for that equipment by the user concerned, although for practical purposes this calibration may then be considered to be applicable to any concrete with natural aggregates not in excess of 20mm maximum size. The Internal Fracture Test may provide a valuable indicator of the general order of insitu strength of an 'unknown' concrete, and be useful when used comparatively. The method is particularly suitable for slender members, although reinforcement and cracked areas must be avoided.

Despite the greater depth of test, more positive load transfer device, and greater cost, the accuracy obtained from the Lok-Test does not fully reflect the apparent sophistication of the method. Excellent agreement has however been found between calibrations from several sources which may be used for 'general' application to concretes with natural aggregates with maximum size of 20mm or less. Applications will principally be related to strength development monitoring in conjunction with pre-established acceptance limits related to curing, shutter stripping, post-tensioning, loading, or quality control specifications. The technique has generated particular interest in the USA in relation to cooling tower construction[11].

The Capo-Test provides a useful way of applying the advantages of the Lok-Test to existing concrete, although the technique is rather slow and relies heavily upon operator skill during the preparatory stages. Although accuracies similar to the Lok-Test are claimed, it is to be expected that scatter will be increased as a result of the drilling and cutting. The accuracy of insitu strength prediction is certainly likely to be better than that achieved with the Internal Fracture Test, but whether this increase will be of sufficient practical significance to justify the greater cost and complexity of procedures is a matter for consideration.

6.0 CONCLUSIONS

Proposed tests fall into two basic categories; those which involve an insert which is cast into the concrete, and those which offer the greater flexibility of an insert fixed into a hole drilled into the hardened concrete. Cast-in methods must be preplanned and will be of use in testing new construction, whilst drilled hole methods will be more appropriate for field surveys of mature concrete. In both cases a particularly attractive feature is that relationships between pull-out forces and concrete strengths are relatively unaffected by mix characteristics and curing history. Test configuration and loading technique however have a significant influence upon such relationships and upon the variability of the test results.

Several individual tests are required to provide an average value relating to the concrete properties at a particular location whichever method is employed, and the accuracy of insitu strength prediction cannot be considered as good unless specific calibrations are available for both the loading technique and mix characteristics involved. Calibrations for general use with unspecified mixes must be developed by the user for his particular equipment and procedures when the Internal Fracture Test is to be used, but this does not appear to be so essential for the Lok-Test.

The principal use of the methods will be for situations in which an indication of concrete strength is required and other forms of non-destructive testing are inappropriate whilst the damage and delays due to core-cutting are unacceptable. In such cases, pull-out tests provide an alternative yielding immediate results with damage confined to the surface zone of the member.

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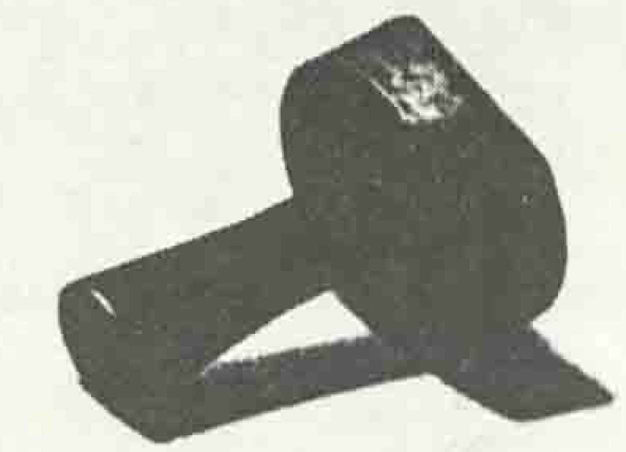

Fig. 1. Lok-Test Insert

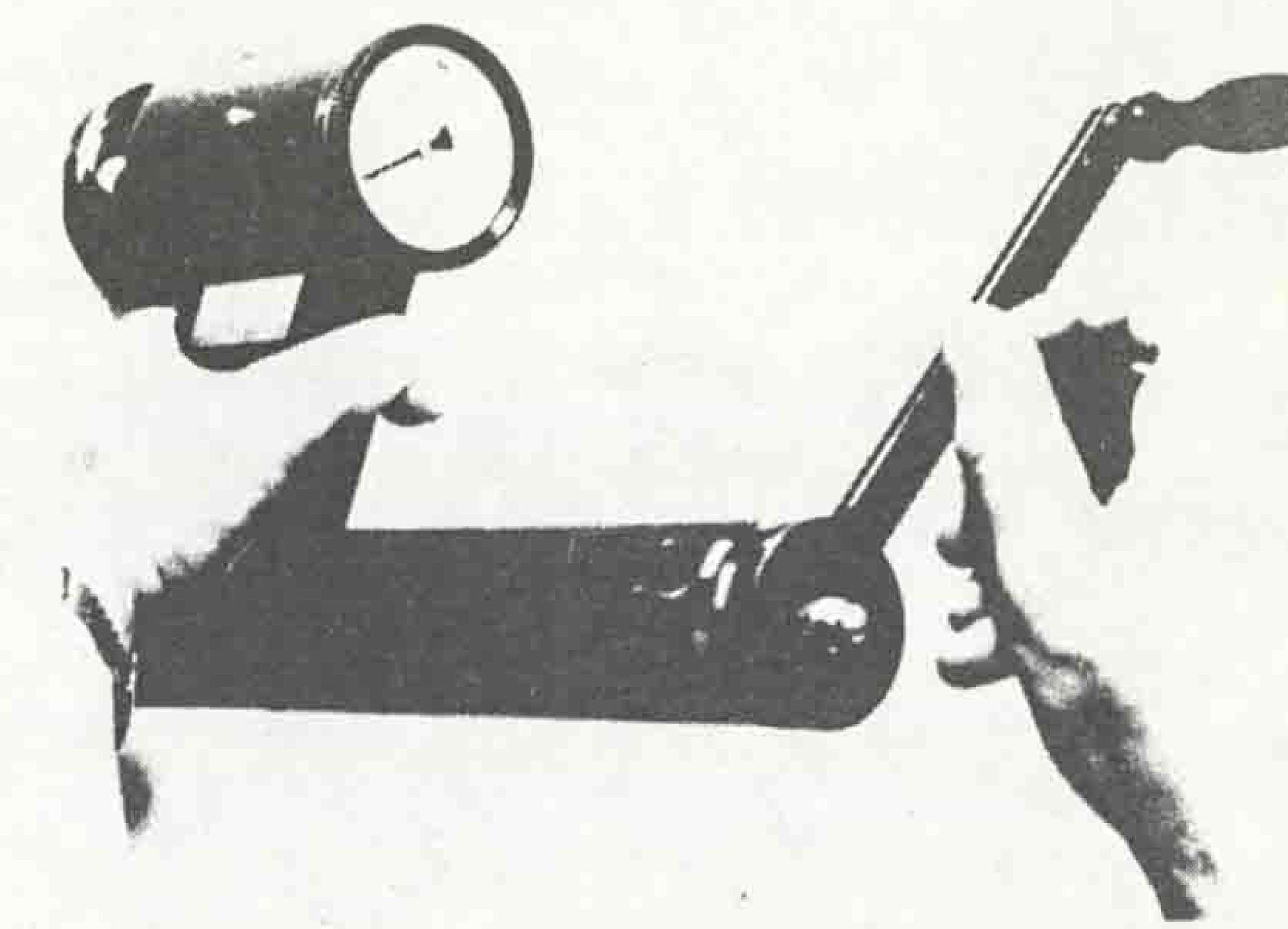

Fig. 2. Lok-Test Jack

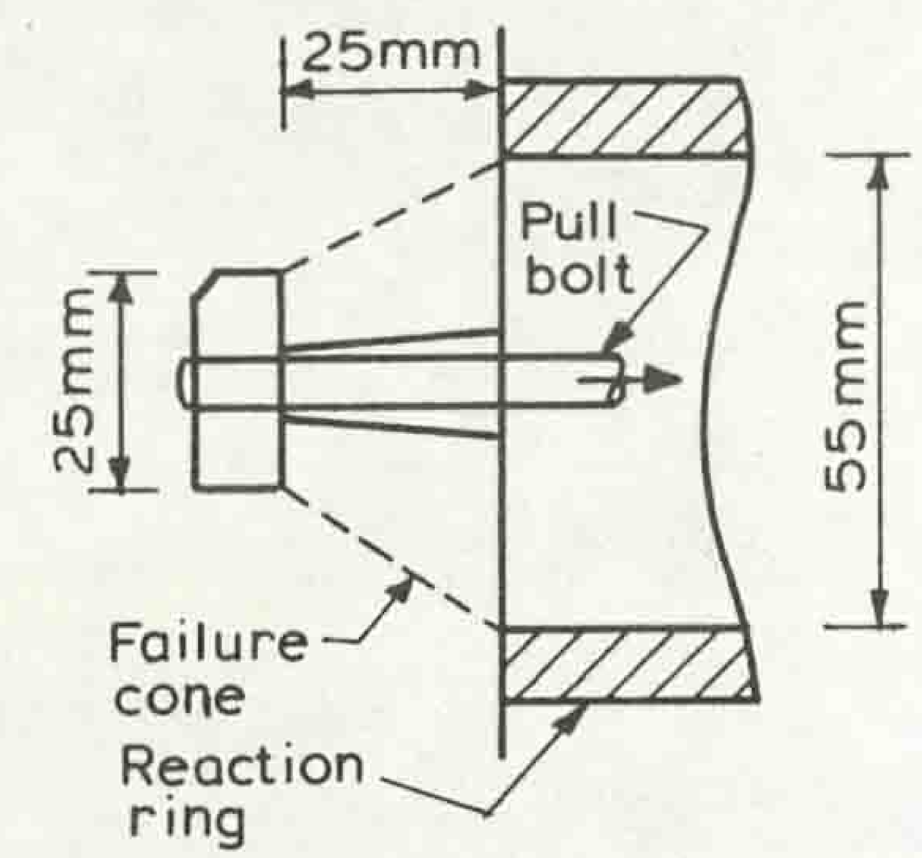

Fig. 3. Lok-Test Configuration

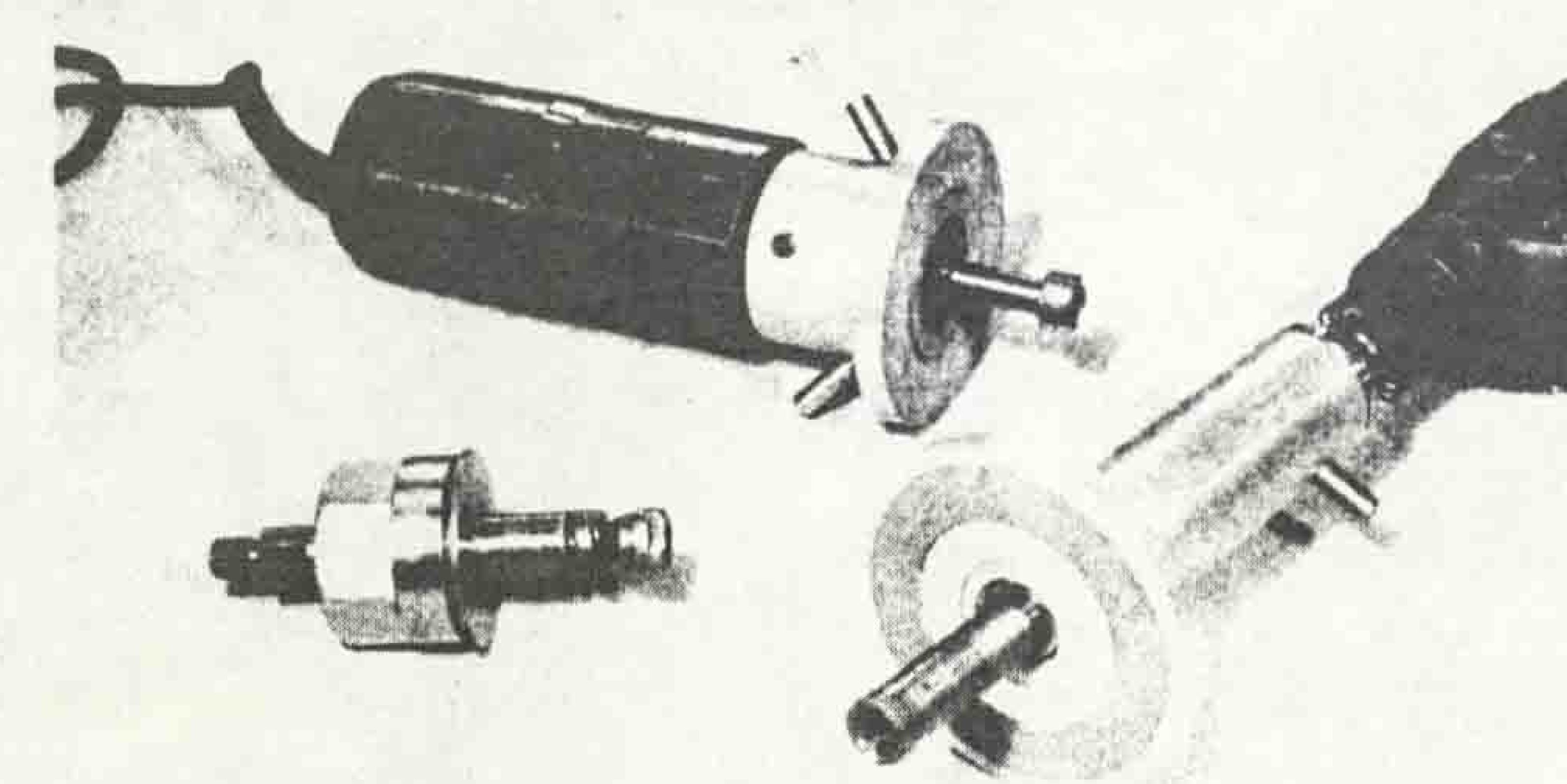

Fig. 4. Capo-Test Equipment

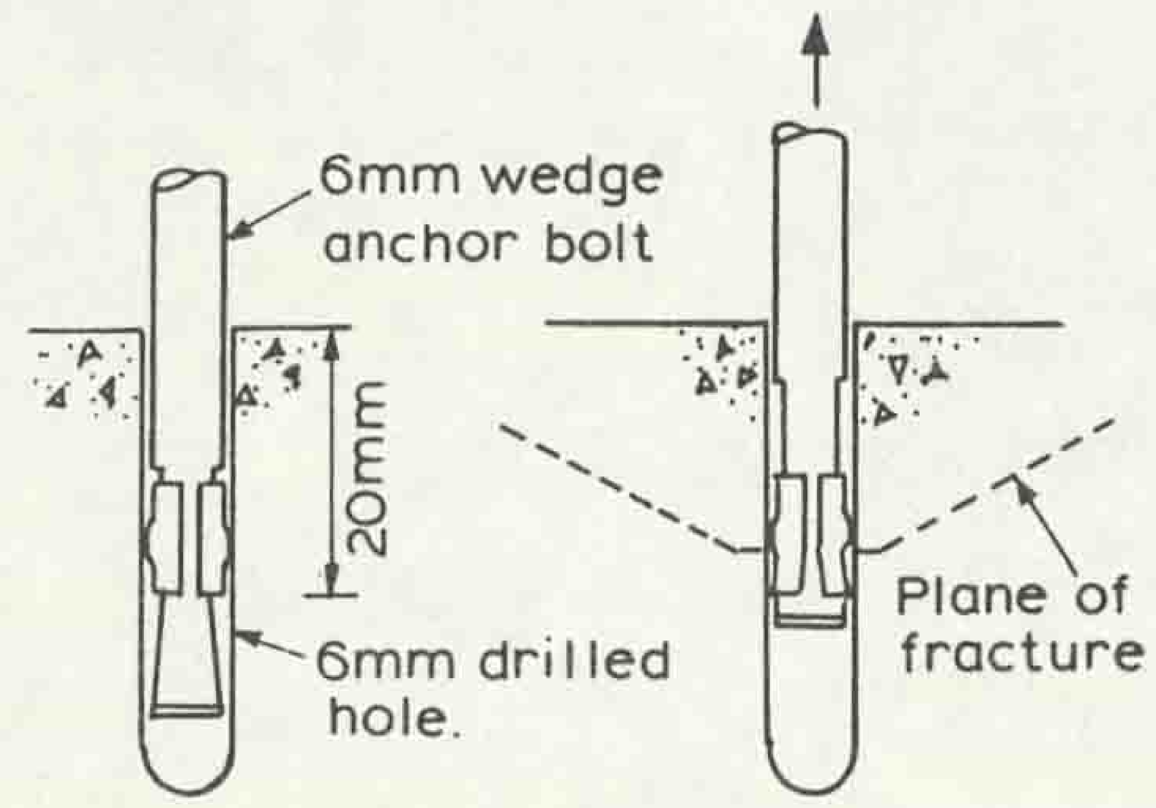

Fig. 5. Internal Fracture Test Configuration

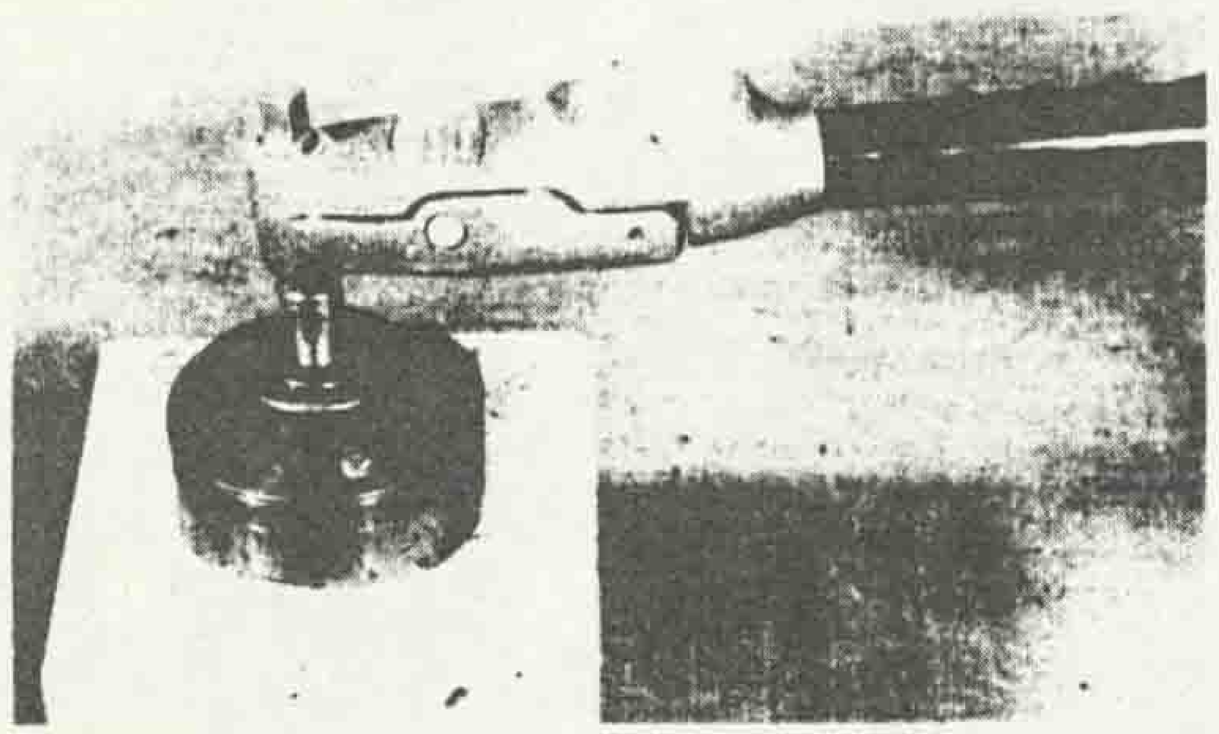

Fig. 6. Torquemeter Method

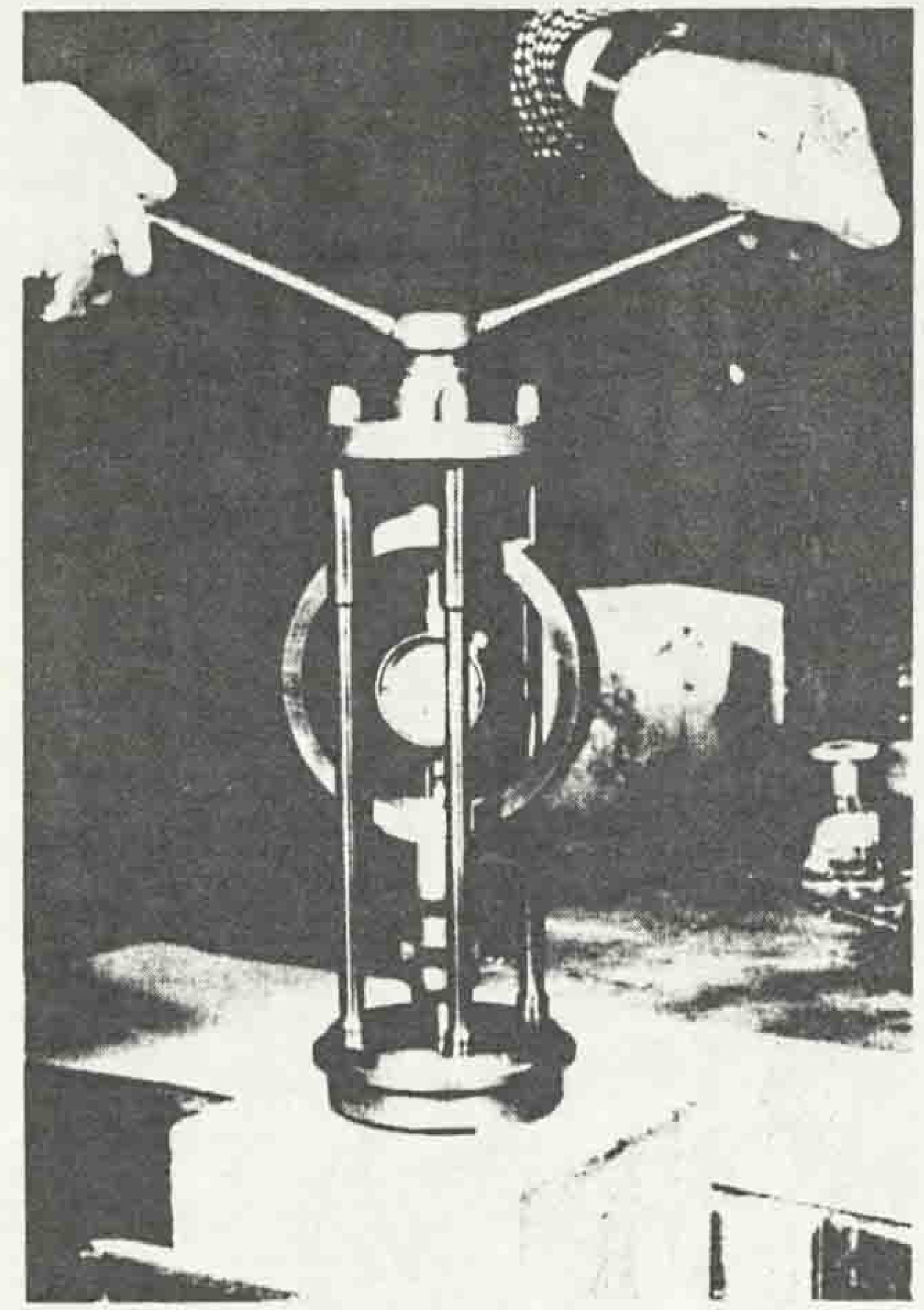

Fig. 7. Direct Pull Method

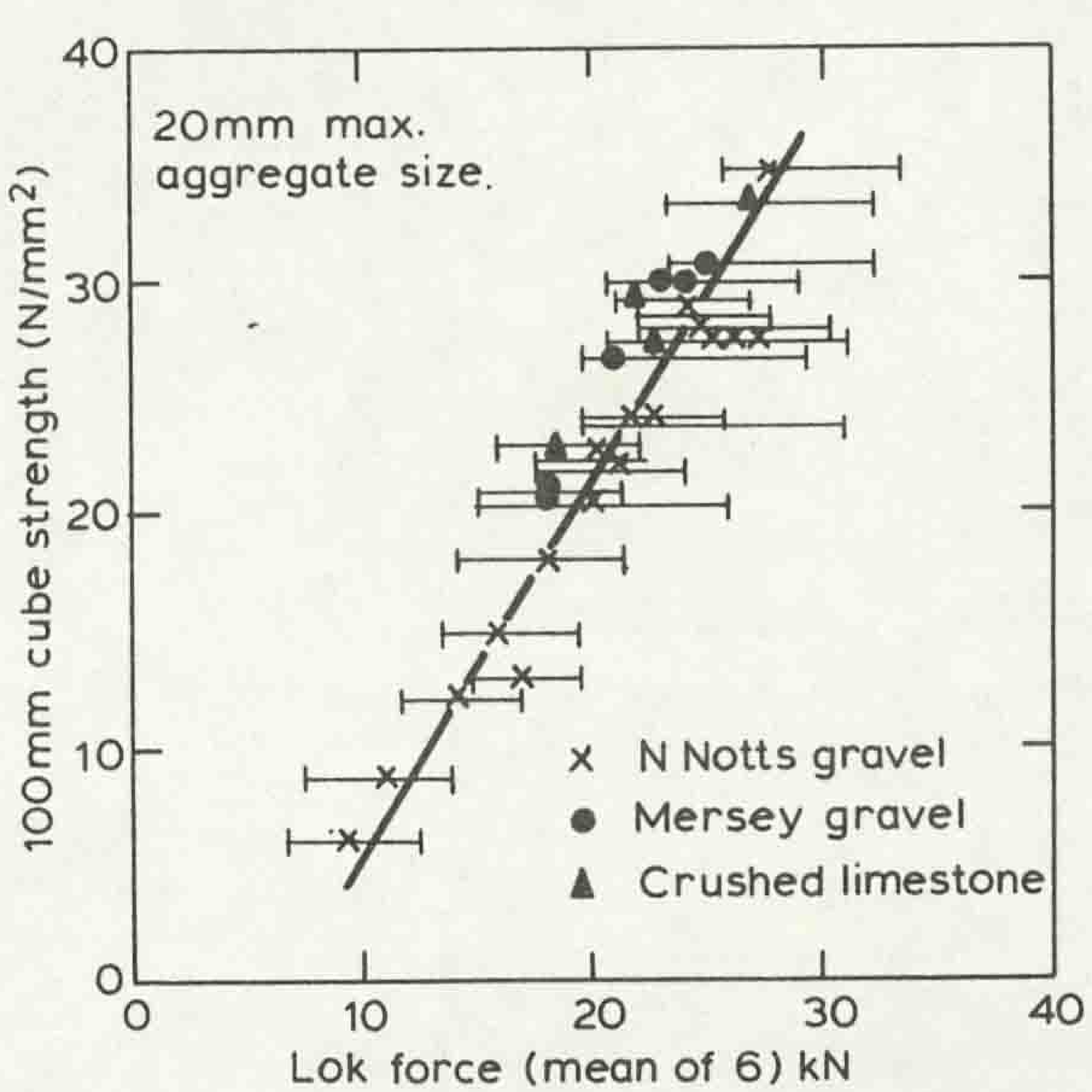

Fig. 8. Lok-Test General Calibration

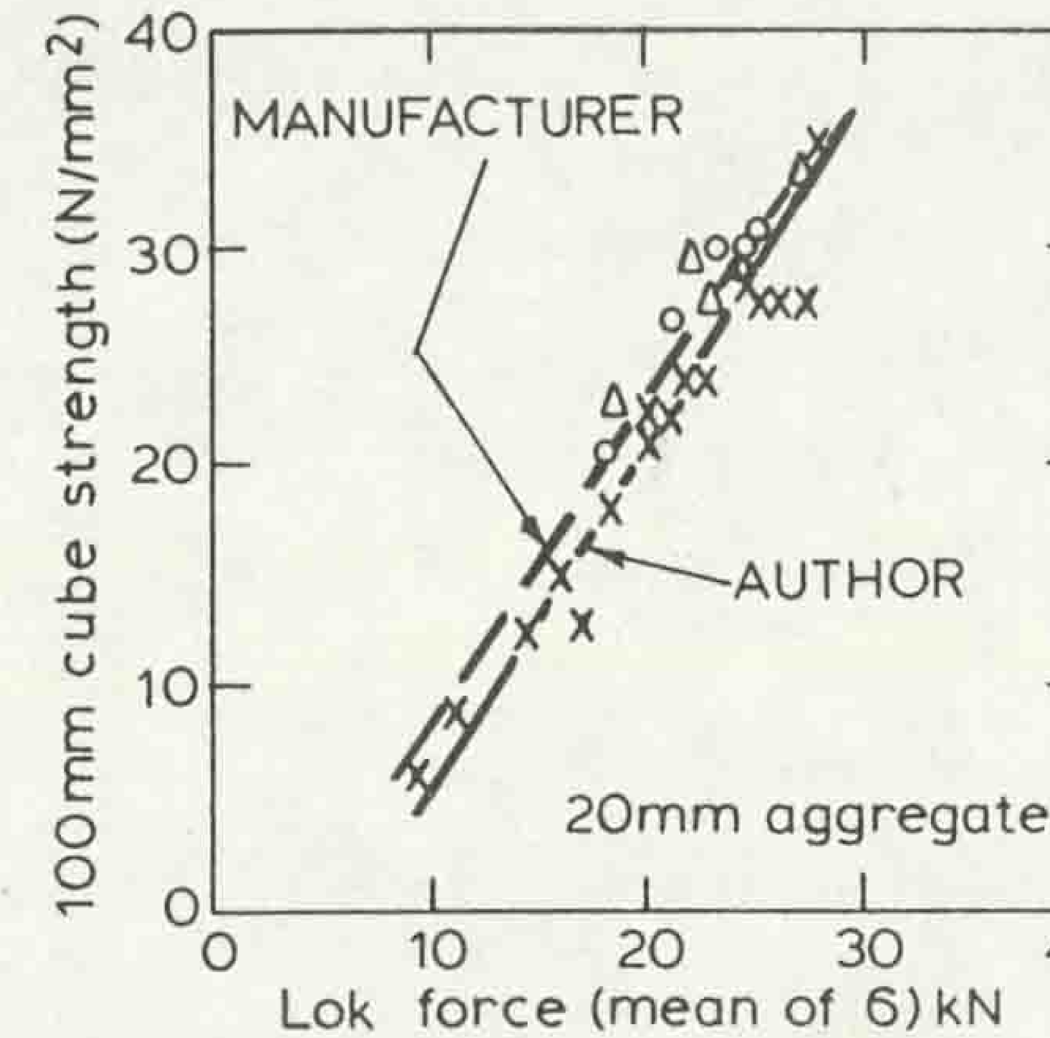

Fig. 9. Lok-Test Calibration Comparison

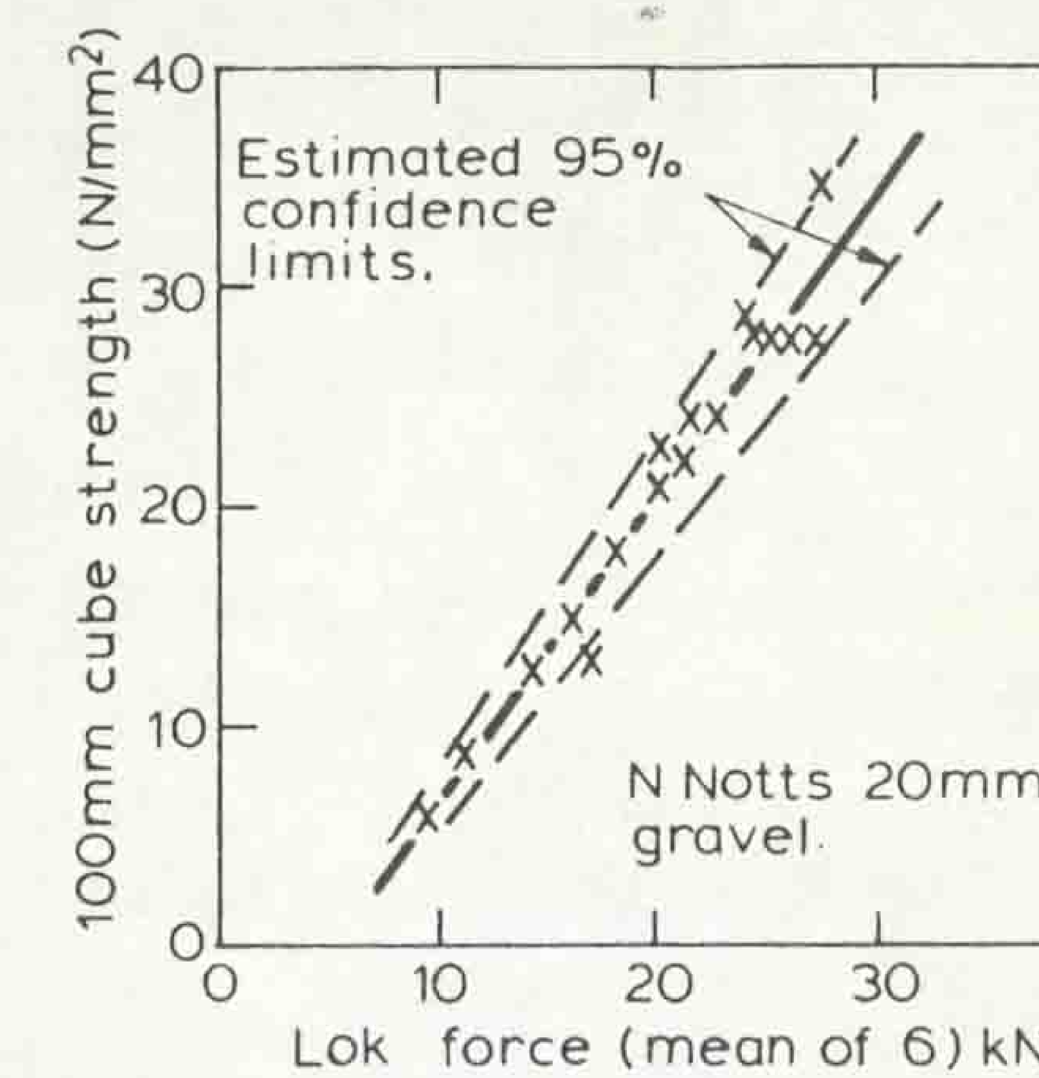

Fig. 10. Lok-Test Calibration for specific aggregate

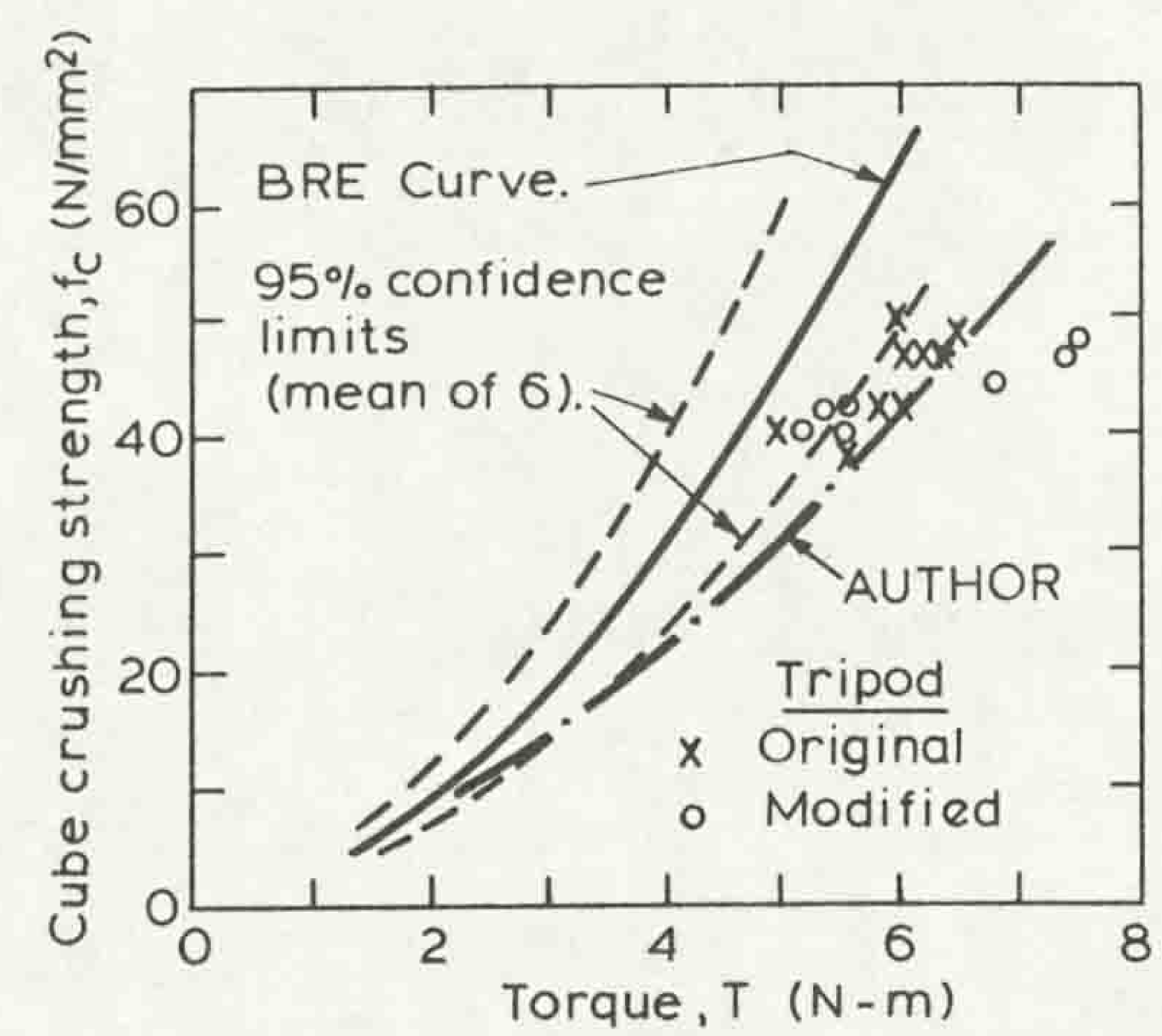

Fig. 11. Internal Fracture Test Calibration Comparison for Torquemeter Method

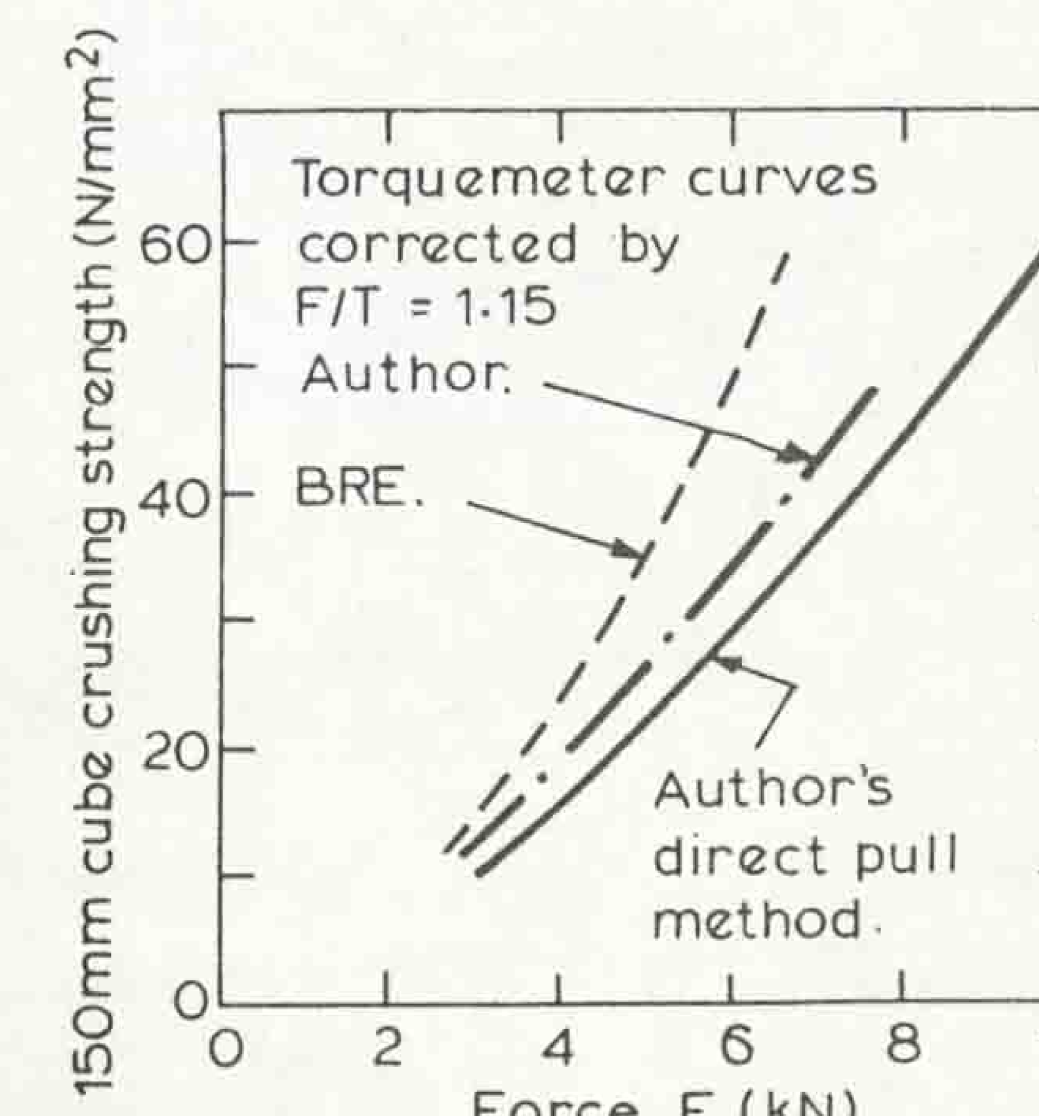

Fig. 12. Internal Fracture Test Force Calibration

Paper 14

"Concrete Strength Determination by Pull-out Tests on wedge anchor bolts"

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Discussion 73 March 1982 pp.249-252

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8442 Concrete strength determination by pull-out tests on wedge-anchor bolts

J. H. BUNGEY, MSc, DIC, MICE*

The Paper describes experimental work undertaken in the laboratory to extend the strength calibration data available for pull-out tests on 6 mm expanding wedge-anchor bolts. The effect of loading technique was examined and a mechanical direct-pull test apparatus was developed. The results are compared with those obtained by torquemeter, and the reliability of strength prediction is assessed with special reference to aggregate type and size. The influence of flexural stress was examined by testing, at various levels on their side faces, two 4·8 m reinforced concrete beams, loaded and unloaded. Using the results the reliability of tests on full-scale members is compared with that of tests on standard laboratory control specimens. The accuracy of the test method is compared with that of other methods available for determining the strength of in-place hardened concrete, and suggestions are made concerning worthwhile applications.

Introduction

The concept of in situ concrete strength determination by measurement of the force required to extract a bolt or similar device embedded in the surface of a concrete member has been proposed for many years but has received increasing attention recently. Early reports were published in the USA in 1938[1] and in the USSR in 1939[2] describing tests in which cast-in bolts were pulled from the concrete. Major developments of this approach occurred in the 1960s, resulting in the Danish Lok test,[3] and a similar method developed by Richards[4] subsequently received considerable attention and led to the publication of a tentative ASTM Standard in 1979.[5] In both cases a flanged insert is cast into the surface of the concrete, and a bolt is subsequently screwed into this insert at the time of test. The bolt is then pulled against a circular reaction ring by means of a hydraulic jack, as indicated in Fig. 1. It has been suggested[6] that failure is produced by crushing of a compressive concrete strut that develops between the flange and the reaction ring, and is thus a direct measure of compressive strength. While such techniques are gaining in popularity as a quality control test, especially in Scandinavia and in the USA, application is limited to preplanned usage, since the pull-out devices must be placed at the time of casting.

2. More flexible proposals have included bolts grouted into drilled holes,[7] bolts with expanding jaws engaging in underreamed drilled holes,[8] and more recently the Capo test,[9] in which a disc is expanded into a slot milled from a drilled hole. However, a much simpler method, involving an expanding wedge-anchor bolt which is inserted into a drilled hole, was proposed by the Building Research Establishment (BRE) in 1977.[10] As the bolt is pulled the load is transferred to the concrete (see Fig. 2), with the reaction provided by a circular

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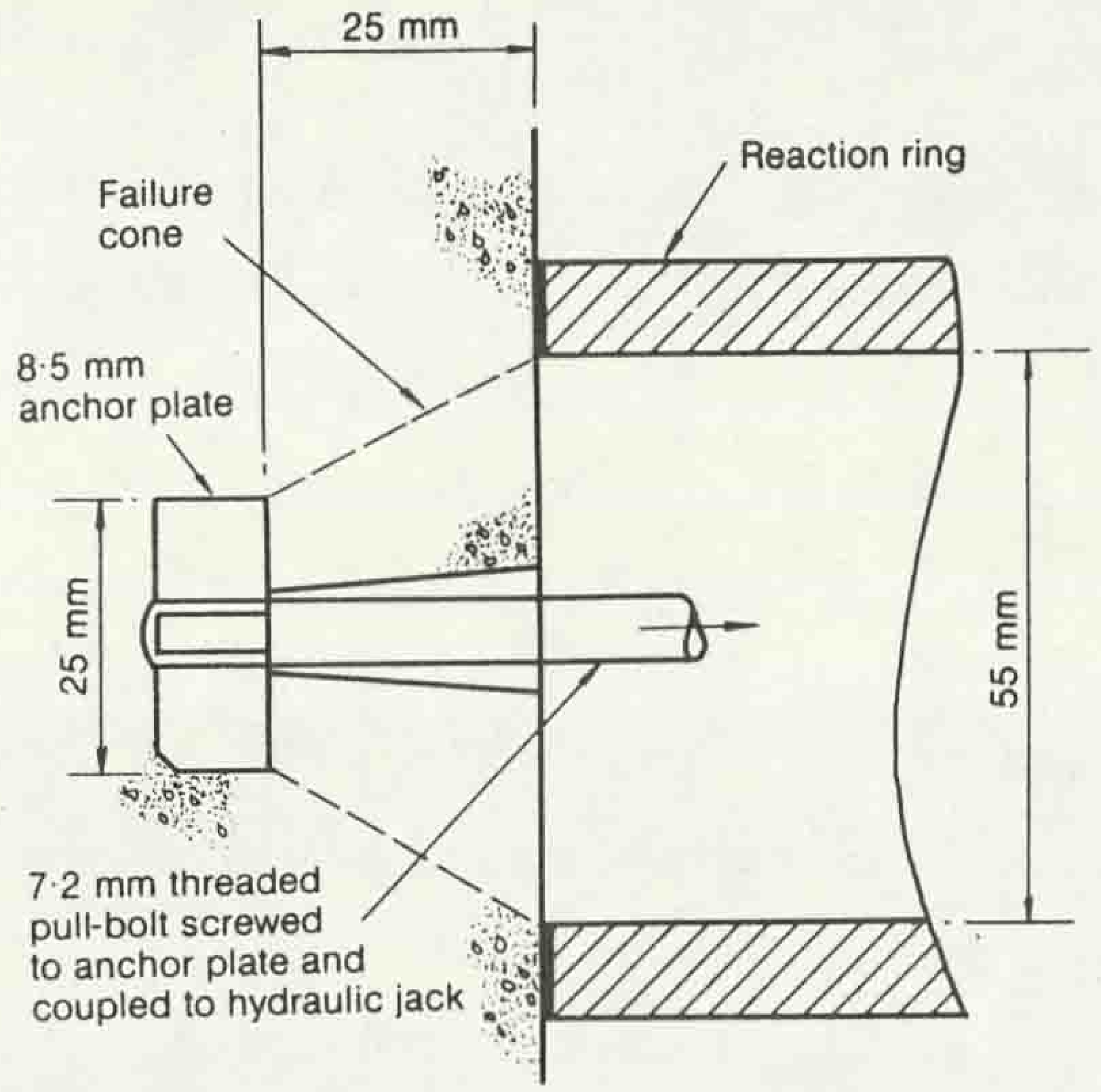

Fig. 1. Lok test arrangement

tripod stool. The concentrated nature of the load transfer between bolt and concrete will result in a more complex failure and suggests that localized influences are likely to have greater effect than on a cast-in insert. Nevertheless, the need for direct non-destructive strength evaluation justifies examination of this approach in greater detail. The technique has been used on a limited scale in the UK, and if problems of test procedure and strength calibration can be overcome, the potential advantages of this simple method are considerable.

Aims and scope of investigation

3. In an effort to assess the influence of some likely variables, on both strength calibration and the reliability of results, laboratory tests were undertaken on a variety of mixes. The principal factors examined were loading technique, aggregate type and size, and stress conditions, although the influence of other mix characteristics and surface hardening was considered.

4. The procedures proposed by the BRE[10] incorporate load application by means of a torquemeter, and basic calibration tests were carried out using this method to enable direct comparison with previously published results. However, subsequent tests were made with a mechanical direct-pull apparatus developed by the Author with a view to overcoming some of the difficulties associated with the torquemeter method. Attention was directed towards the relationship between calibrations for the two loading methods, and the variability of results in each case, to enable an assessment of the likely accuracy of strength predictions based on this approach.

5. The published data relating to Portland cement concretes subjected to this form of testing are limited and in this investigation 10 mm and 20 mm gravels and granites were used to provide additional calibration data. The calibration tests were carried out on 150 mm laboratory cast cubes. In addition to mix characteristics it was considered that in practice the stress state of the concrete at the time of test was potentially the most significant of other possible

variables, and two 4800 mm × 500 mm × 200 mm reinforced beams were also cast and tested while under load to assess the influence of this effect.

6. A number of theoretical approaches involving tensile and shear strength have been proposed to describe the failure mechanisms of pull-out tests.[11] However, no real advantage is to be gained from a detailed theoretical analysis from the point of view of practical application of the test method in a standardized form, especially in view of the observed dependence of measured results on the load application technique and rate. This investigation has therefore concentrated on experimental observations to provide empirical strength/pull-out force relationships and to assess the influences of the variables considered.

Basic test procedures

7. For convenience, the basic procedures proposed by the BRE[10] have been used throughout this investigation. A hole is first drilled 30–35 mm deep into the concrete using a roto-hammer drill with a nominal 6 mm diameter bit. The hole is then cleared of dust with an air blower, and a 6 mm bolt with an expanding sleeve is tapped lightly into the hole by hammer until the sleeve is 20 mm below the surface, as shown in Fig. 2. Verticality of bolt alignment relative to the surface can be checked using a simple slotted template.

8. The bolt is loaded at a standardized rate against a tripod reaction ring 80 mm in diameter, with three feet, each of which is 5 mm wide and 25 mm long. If necessary, shims may be used to correct for minor bolt misalignments. After applying an initial load to cause the sleeve to expand, the force required to produce failure by internal fracture of the concrete is measured. This will be the peak load indicated by the typical load/movement pattern in Fig. 3. If the load is reduced once this peak has been reached there is likely to be no visible surface damage and it has been suggested that the bolt can be sawn off. If load applica-

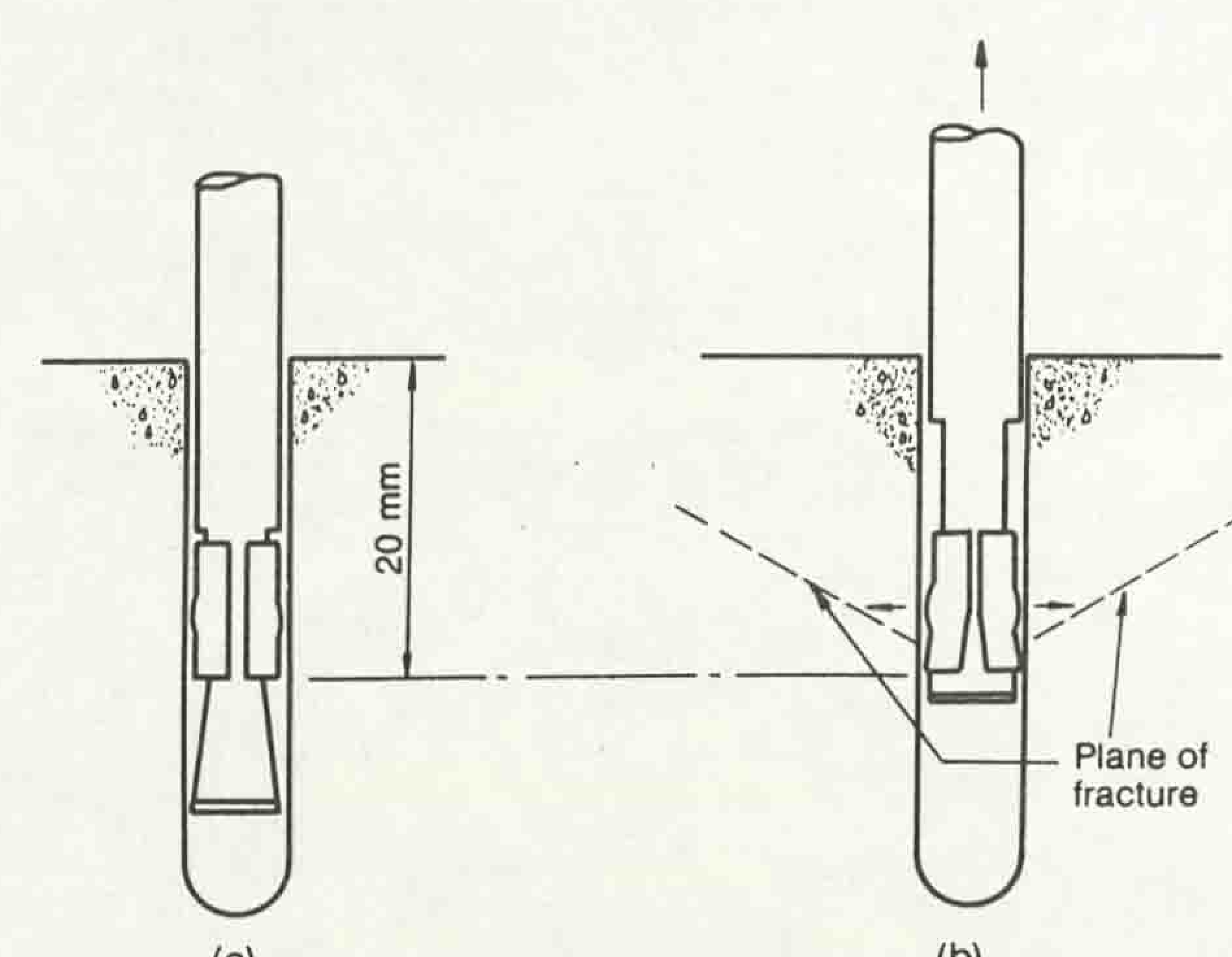

Fig. 2. Pull-out of wedge-anchor bolt: (a) bolt inserted into drilled hole; (b) anchor under load

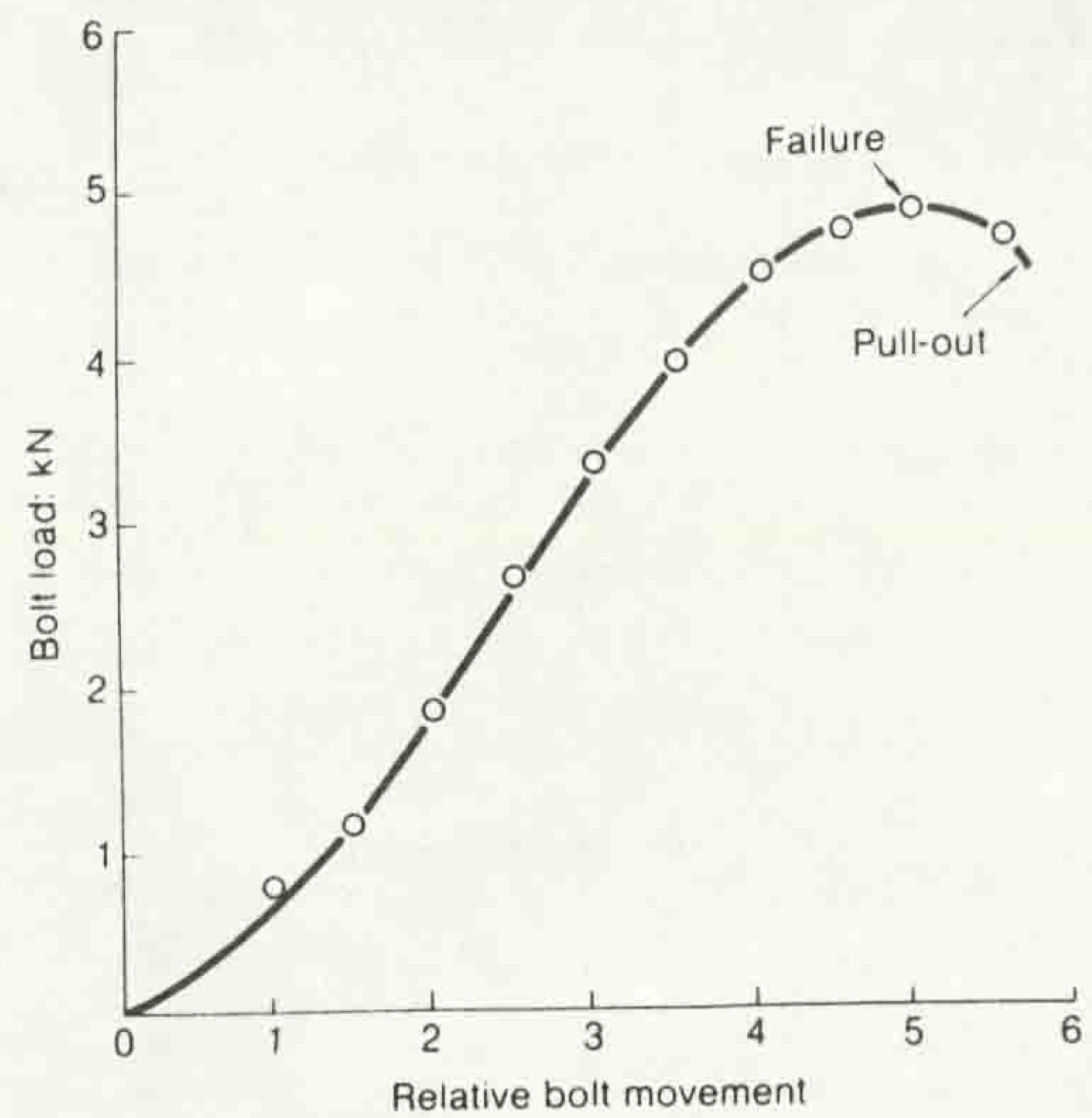

Fig. 3. Typical bolt behaviour

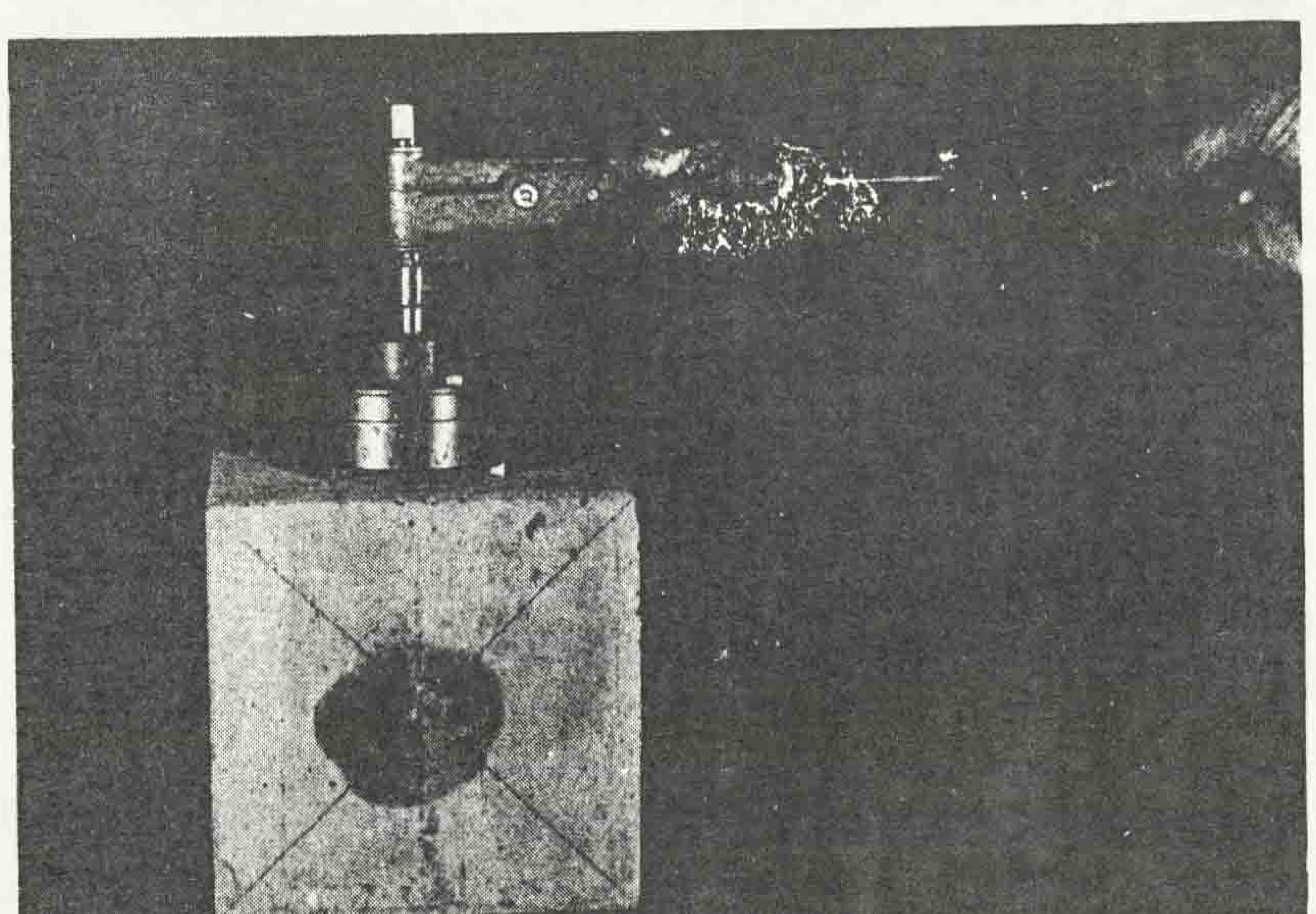

Fig. 4. Torquemeter loading method

tion continues a cone of concrete will be pulled from the surface, often intact, and considerable making good may be necessary.

Loading methods

9. The BRE recommend that load is applied through a nut on the greased bolt thread by means of a torquemeter which is rotated one half-turn in 10 s and released before reading, with the procedure being repeated until a peak is passed. This equipment is shown in Fig. 4. Ash[12] has indicated, however, that the bolt is subjected to torque as well as to direct force, resulting in failure at a lower load than for a direct force alone. It would thus seem sensible to reduce the variables by providing a straight pull to the bolt. Ash proposed the use of a hydraulic jack, and this is also the recommended method for cast-in tests in the USA,[5] although these tests are of a larger scale than the method under investigation. However, the Author's observations on the use of this approach under site conditions on 6 mm bolts have indicated difficulties of standardized load application using lightweight equipment, coupled with insensitivity of load recording.

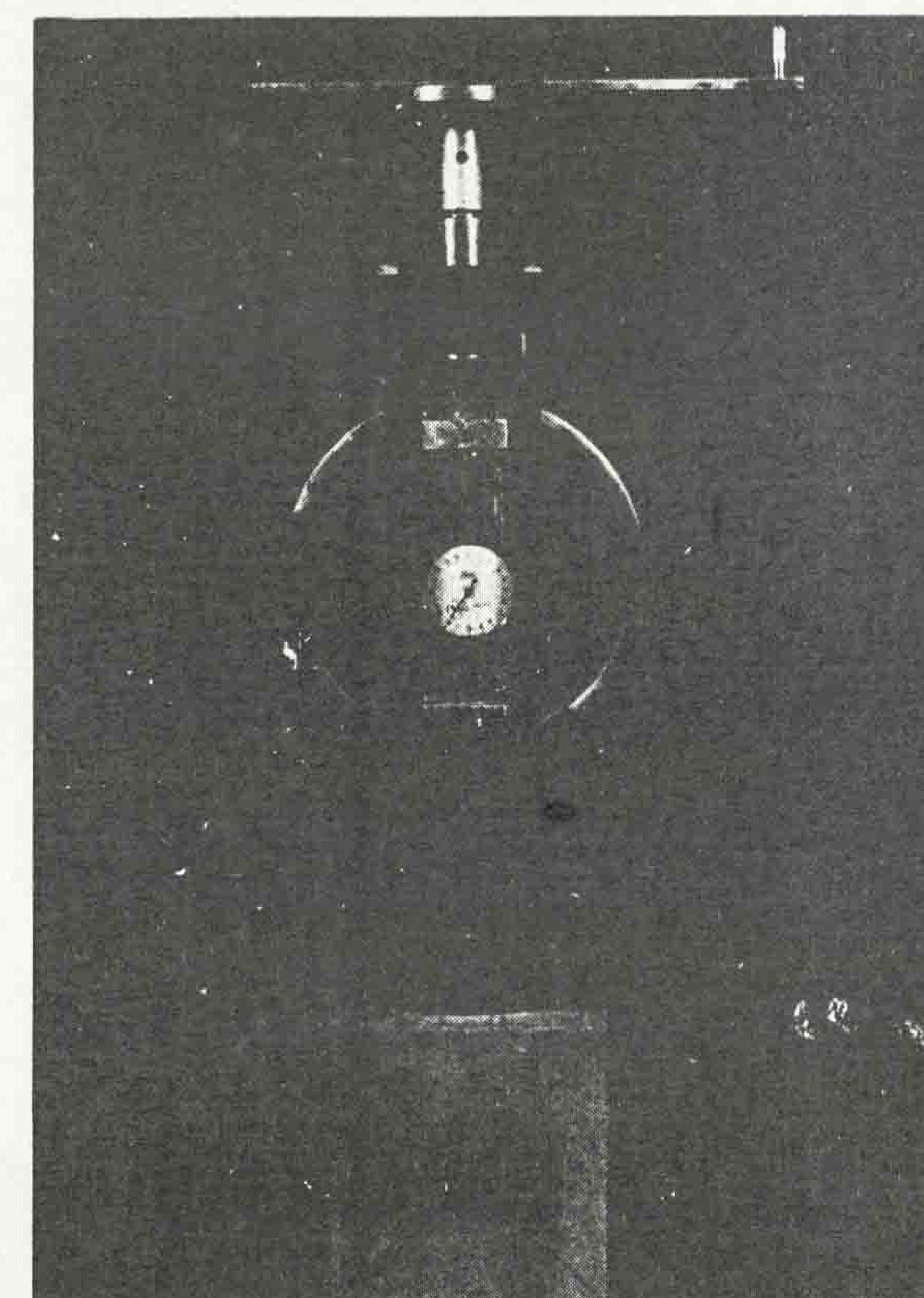

Fig. 5. Author's loading method

This makes observation and recording of the peak value difficult and increases the likelihood of surface damage due to continued load application.

10. It was felt that a mechanical method was likely to suffer less from these disadvantages, and so the equipment shown in Fig. 5 has been developed. This apparatus has an identical reaction tripod to that used with the torquemeter and is fixed to the projecting bolt by means of an adaptor nut. Load is applied by rotating the loading handle steadily at a standard rate, with the current load indicated by a dial gauge mounted on a proving ring. The peak load can thus be easily and accurately observed. There is sufficient flexibility in the equipment to accommodate minor variations of bolt alignment, and in addition to giving a direct pull this equipment is cheap, portable and very sensitive. A later version has been developed which is more compact and has modified twin loading handles, but this was not used in tests reported here.

Details of test programme

11. The mixes used in the investigation are shown in Table 1, and incorporate either an angular crushed granite or irregular North Notts gravel aggregate with sand fines. Initially, calibration tests were carried out using mixes 1–4 with the loading applied by torquemeter. Pull-out tests were made on the four side faces of 150 mm cubes, which were cured at 100 % relative humidity and 18–20 °C, but allowed to dry for two to four hours before testing. Tests were performed at 3, 7, 14 and 28 days, and companion 150 mm cubes were used in groups of three for compressive strength testing in accordance with BS 1881.[13]

12. Preliminary tests with the Author's equipment were undertaken on a group of similar cubes from mix 2, to examine the effect of rate of load application. The intention was not to examine this variable in detail, but to establish a convenient standard loading rate for future work. Subsequently 150 mm

Table 1. Mix details

Mix	Aggregate		Cement type*	Aggregate/cement	Water/cement
	Max. size, mm	Type			
1	20	Granite	RHP	5·3	0·65
2	20	Gravel	RHP	5·0	0·50
3	10	Granite	RHP	5·3	0·73
4	10	Gravel	OP	7·7	0·76
5	10	Gravel	RHP	7·3	0·76
6	10	Gravel	RHP	7·3	0·60
7	10	Gravel	RHP	6·1	0·52
8	10	Gravel	RHP	7·5	0·87
9	10	Gravel	RHP	7·5	0·70
10	10	Gravel	RHP	7·4	0·90
11	10	Gravel	RHP	7·4	0·65
12	10	Gravel	RHP	6·9	0·85
13	10	Gravel	RHP	5·3	0·60

* RHP—rapid hardening Portland cement; OP—ordinary Portland cement.

Table 2. Comparison of sound and damaged 150 mm cubes

Mix	Age, days	Mean strength of three damaged cubes, N/mm^2	Mean strength of three sound cubes, N/mm^2	Percentage reduction
1	14	26·9	28·3	4·9
1	28	33·9	35·4	4·2
3	28	51·3	53·3	3·8

cubes from mixes 8–13 were tested on two opposite faces with each set of loading equipment to permit a direct comparison of results. More detailed calibrations for the direct pull method were then produced for 10 mm gravel mixes 5–13, which cover a range of aggregate/cement and water/cement ratios and were tested at either 14 or 28 days. A number of the cubes that had been subjected to pull-out tests on four faces, and were thus damaged, were also tested in compression and compared with corresponding sound specimens. The results are summarized in Table 2. Although the average reduction of 4·3% is small, and appears to be reasonably consistent, it was considered preferable to base calibrations on the strengths of sound cubes.

13. The beams used to assess the effects of flexural stress were cast from 10 mm North Notts gravel concrete (mix 5), cured in the laboratory under damp hessian for 7 days and then dried at approximately 18 °C before testing after 28 days. The beams were subjected to third point loading from hydraulic jacks in a testing frame, and pull-out tests were taken horizontally at four levels on each face in the middle third of the 4·5 m span using the Author's equipment. Strain readings were also taken at six levels with a 200 mm Demec guage and the dynamic modulus was obtained by electrodynamic tests on 500 mm × 100 mm × 100 mm prisms in accordance with BS 1881.[14] Control cubes and prisms were cured in a similar manner to the beams and tested at the same time as the beams. Tests were first made with the beams carrying no load other than their own self-weight. Beam A was then loaded to keep the flexural stresses within the uncracked elastic range and further tests were taken while under sustained load. Beam B was subjected to a higher loading intensity and exhibited extensive flexural cracking at the time of testing. Twenty-four pull-out tests were made on each face before loading, and another 24 when under load. Deflexions were also monitored at the third points for both faces of the beams to check for uniformity of loading.

Experimental results

Influence of aggregate properties on strength calibration

14. Results for tests with the torquemeter equipment on mixes 1–4 are shown in Fig. 6, which relates to 10 mm aggregate, and in Fig. 7 for 20 mm aggregates. In both cases the results are expressed in terms of the average compressive strength of three sound 150 mm cubes, and each point represents the mean of four pull-out tests on an individual cube from the same batch. In neither case is it possible to differentiate between aggregate types. The relationship for Portland cement concrete proposed by the BRE,[10] which is based on 100 mm cubes, has been corrected to equivalent 150 mm cube strengths by a reduction of 4 %[15] and is also indicated in both figures. It has been suggested[10] that the results will follow a log-normal distribution, and regression analysis of

each set of results yields the relationships listed in Table 3. The combined curve for each aggregate size has also been drawn on the appropriate figure. It is apparent that aggregate size does not significantly affect the strength/pull-out force relationship, which in both cases is below the BRE curve.

15. Comparison of Figs 6 and 7, and of the correlation coefficients obtained for the curves, shows clearly that the variability of results with 20 mm aggregate is greater than for 10 mm maximum size.

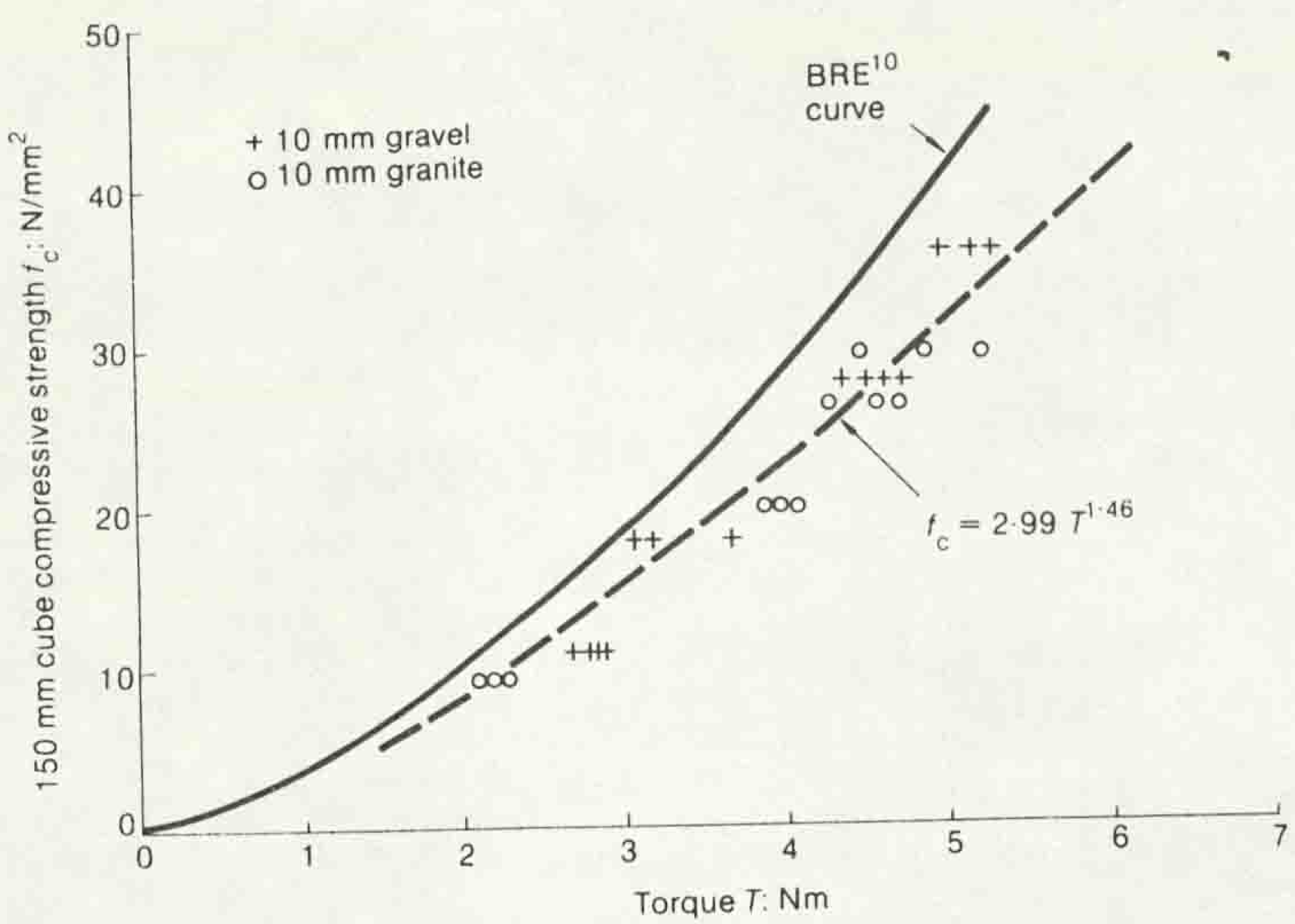

Fig. 6. Pull-out calibration for 10 mm aggregates by torquemeter method

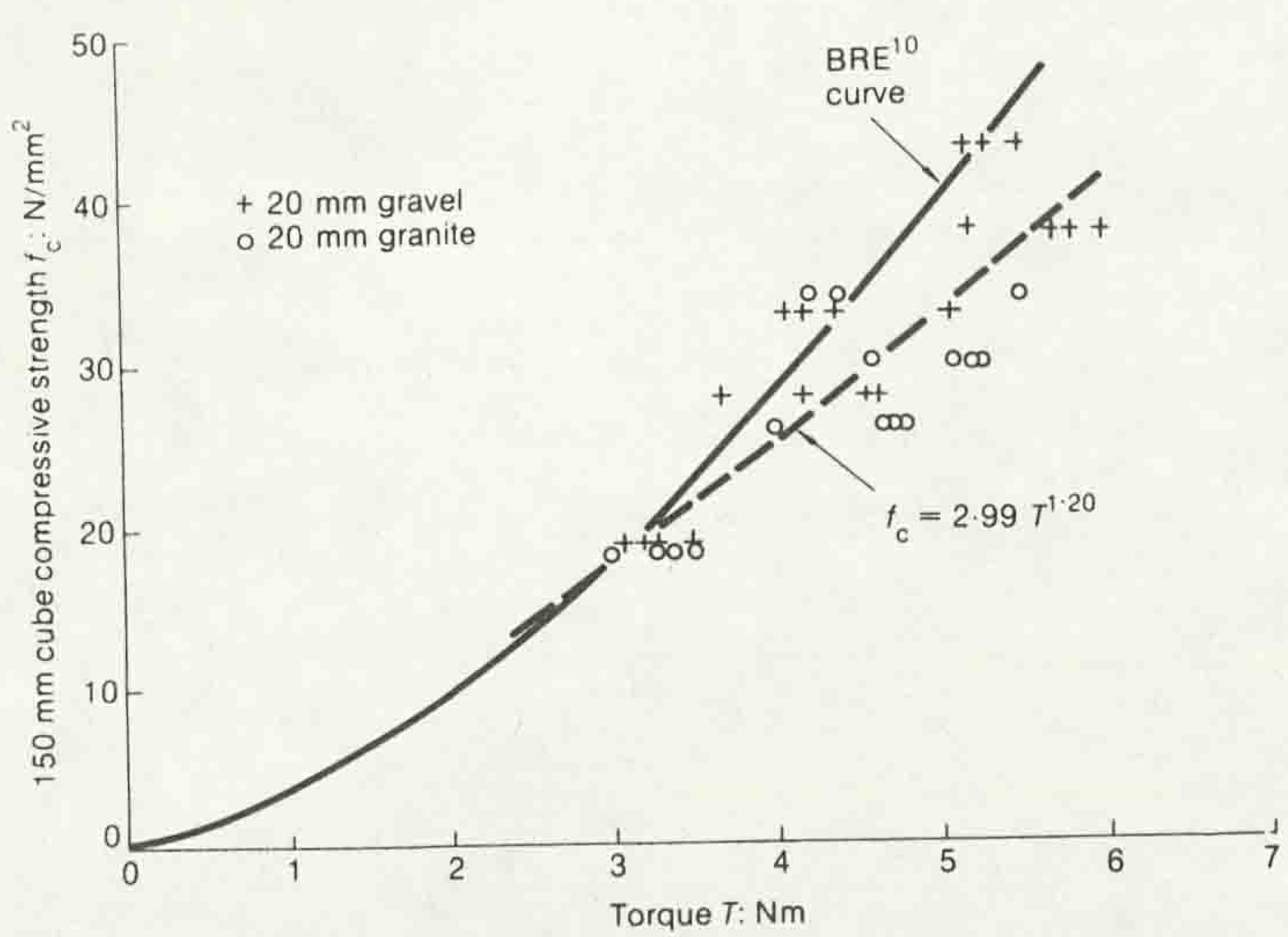

Fig. 7. Pull-out calibration for 20 mm aggregates by torquemeter method

Influence of loading technique

16. The rate of load application was found to have a considerable influence on the magnitude of the measured failure force. A steadily increasing strain was also found to produce more consistent results than where settling time pauses were included, and a rate of one complete revolution in 20 s was found to be convenient for the operator. This was thus adopted as standard for all subsequent work with the Author's apparatus.

17. Results of comparative tests between this and the torquemeter loading technique are summarized in Table 4, which represents the results of three cubes from each of the six mixes. The mean value of force/torque ratio is 1·4, and this relationship is used when comparing calibrations obtained by the two methods.

18. The average coefficients of variation of pull-out tests on 150 mm cubes for both loading methods are summarized in Table 5, and are based on the results of tests on a large number of groups of at least three similar specimens.

Table 3. Calibration relationships

Aggregate		Strength/torque relationship	Correlation coefficient
Maximum size, mm	Type		
10	Gravel	$f_c = 2{\cdot}91T^{1{\cdot}51}$	0·96
	Granite	$f_c = 3{\cdot}03T^{1{\cdot}41}$	0·97
	Combined	$f_c = 2{\cdot}99T^{1{\cdot}46}$	0·96
20	Gravel	$f_c = 4{\cdot}68T^{1{\cdot}25}$	0·82
	Granite	$f_c = 5{\cdot}59T^{1{\cdot}05}$	0·72
	Combined	$f_c = 4{\cdot}79T^{1{\cdot}20}$	0·76

Table 4. Relationship of torque and force

Mix	Mean 150 mm cube strength, N/mm²	Mean pull-out force to torque ratio
8	19	1·55
10	20	1·49
12	21	1·32
9	35	1·25
11	36	1·24
13	42	1·52

Table 5. Coefficients of variation of tests on cubes

Aggregate size	Average coefficient of variation of torque, %	Average coefficient of variation of direct force, %
10	12·4	5·4
20	16·5	7·0

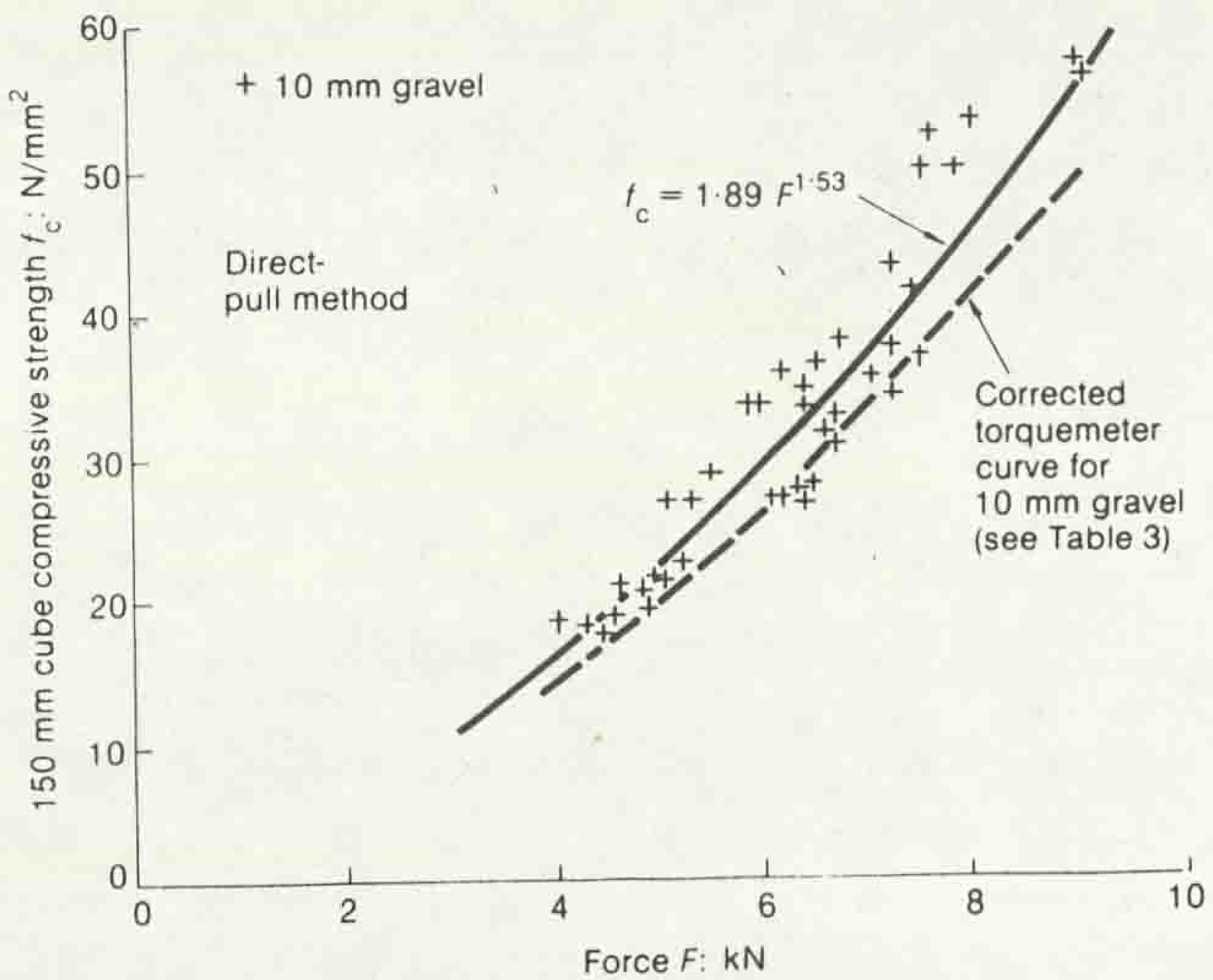

Fig. 8. Pull-out calibration for a range of 10 mm gravel mixes

Influence of mix proportions

19. Results of tests on a wide range of 10 mm gravel mixes (5–13) are shown in Fig. 8, where each point again represents the mean of four pull-out tests on a cube. The curve obtained for these results was found to have a correlation coefficient of 0·90 and is plotted, together with the relationship obtained previously for 10 mm gravel concrete by torquemeter corrected by the mean force/torque ratio already determined. The curve including a range of aggregate/cement ratios lies close to, but above, that for the specific mix.

Influence of flexural stress

20. Concrete properties for the control specimens relating to the beams are summarized in Table 6, with the static modulus derived on the basis of CP 110[16] recommendations. The measured strain distributions are shown in Fig. 9 together with corresponding stress distributions estimated on the basis of the above static modulus values and cube strengths. Earlier Brazilian tests on concrete from this same mix yielded a splitting tensile strength of 2·7 N/mm², which may be expected to correspond to a flexural strength of approximately 4·5 N/mm²,[15] and this has been assumed to be the likely cracking value.

21. Results are presented in Table 7, in which each pull-out value represents the average of 12 readings, including both faces of the beam, at each level. Initial

Table 6. Beam concrete properties

Beam	150 mm cube strength, N/mm²	Dynamic elastic modulus, kN/mm²	Calculated static modulus, kN/mm²
A	32·5	41·2	32·5
B	31·5	37·3	27·7

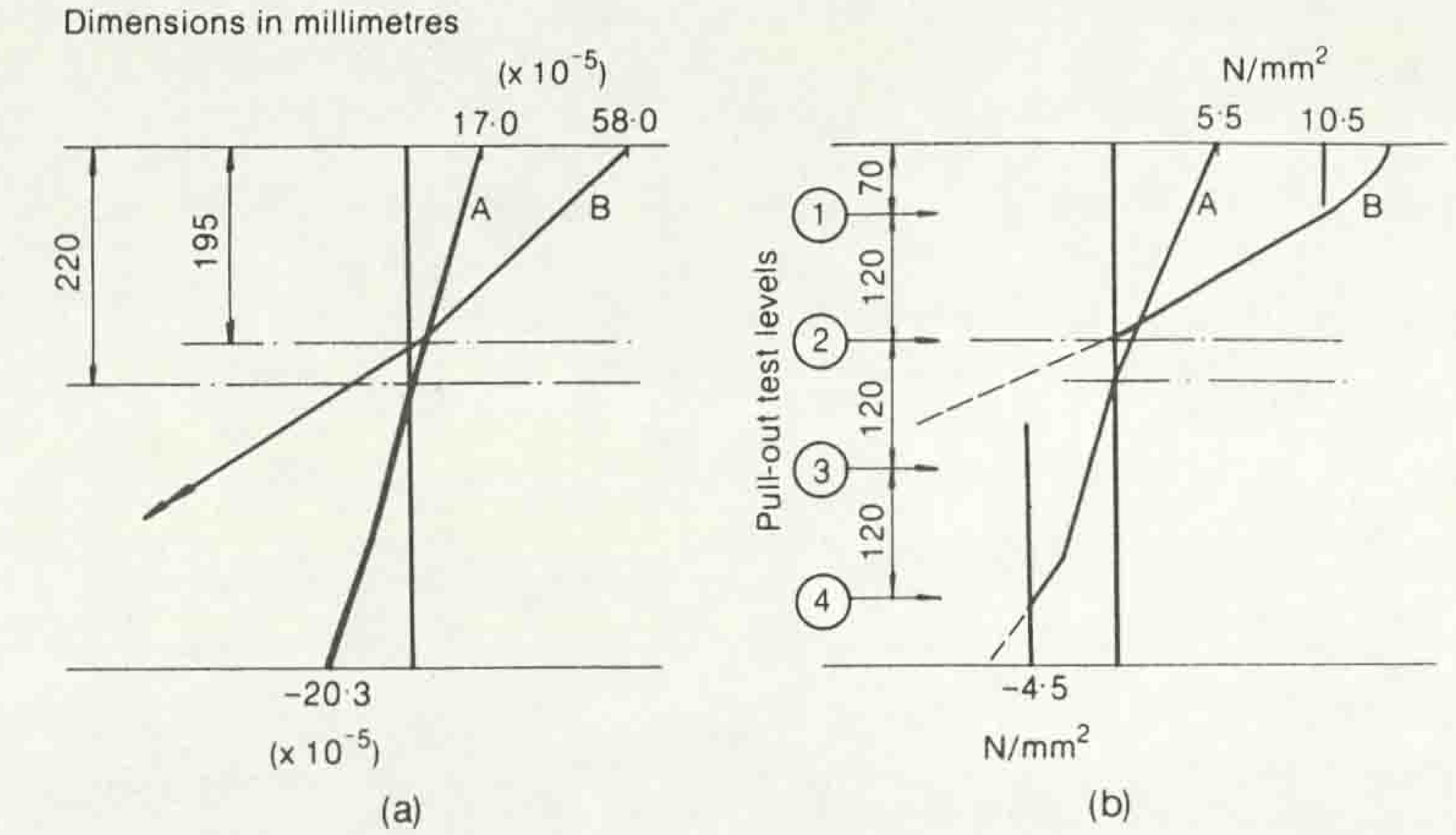

Fig. 9. Beam test flexural strain and stress distributions: (a) strain distributions under load; (b) estimated stress distributions

examination of the results for the unloaded members shows good agreement between the two beams in terms of the distribution of strength and coefficient of variation across the depth of the member. On average beam B appears to be marginally stronger, but the overall variability of results is similar for the two beams. The effects of both compressive and tensile stress appear to be similar and result in a general reduction in pull-out force, although the size of this reduction is so small that it cannot be regarded as statistically significant. However, the presence of visible flexural cracks passing through the region of a pull-out failure cone results in a significant reduction of measured force; these results have been discounted in calculating the average value for the beams. The coefficient of variation shows a marked increase for the tests under loaded conditions, and this surprisingly includes the zones with very low levels of stress. This feature is apparent for both beams, although the pattern is contradicted by the results relating to the most highly stressed condition.

Discussion of results

22. The calibration tests confirm that aggregate type and shape do not appear to be important in terms of the strength/pull-out force relationship, and furthermore suggest that size is similarly unimportant. This latter finding is in agreement with suggestions by Sorensen[17] relating to the Lok test, although the load transfer in that case is more clearly defined and the depth of the test is greater. Examination of many failure cones obtained by continuing loading past the peak shows an average failure depth of 17 mm; thus it is not surprising that mixes using 20 mm aggregate show a greater variability of results than for smaller sizes. This increased variability is demonstrated both by the average correlation coefficients of 0·76 and 0·96 and by the coefficients of variation given in Table 5. While smaller aggregates may be expected to show an even better reliability, the use of the test for aggregates greater than 20 mm is likely to prove unreliable, despite Sorensen's[17] successful use of the Lok test with larger sizes.

23. The force/torque ratio obtained when comparing the two loading techniques is greater than the value of 1·15 reported by the BRE[10] for tests using

Table 7. Summary of beam test results

Beam	Test level	Unloaded		Loaded			Ratio of (loaded/unloaded) values	
		Average pull-out force, kN	Coefficient of variation, %	Estimated stress, N/mm^2	Average pull-out force, kN	Coefficient of variation, %	Pull-out force, kN	Coefficient of variation, %
A	1	5·0	8·5	+3·8	5·2	12·6	1·04	1·5
	2	5·5	10·1	+0·8	5·1	18·0	0·93	1·8
	3	6·8	9·5	−2·0	6·4	13·0	0·94	1·4
	4	7·3	6·3	−4·5	6·8	6·9	0·93	1·1
	Average	6·2	8·6		5·9	12·6		
B	1	5·9	8·7	+10·5	5·8	6·4	0·98	0·7
	2	6·9	10·2	0	6·7	16·2	0·97	1·6
	3	6·8	9·1	Cracked	6·4	10·0	0·94	1·1
	4	8·0	6·5	Cracked	5·9*	—	0·74	—
					7·9	13·6	0·99	2·1
	Average	6·9	8·6		6·7	11·6		

* Cracks within test zone.

the torquemeter, but also incorporating a load cell. This difference is to be expected in view of the anticipated greater resistance to a direct pull and as a result of measurement of peak values; the torquemeter method yields settled values. The observed range of $\pm 0{\cdot}15$ on this mean ratio is comparable with that reported by the BRE and conversion of results from torque to force, or vice versa, should thus be avoided wherever possible. Analysis of the results published by Ash[12] and obtained by a hydraulic direct-pull method suggests a corresponding ratio of about 1·3, although precise details of the loading procedure are not given in that case.

24. The coefficients of variation in Table 5 show a distinct reduction in test variability when using the Author's equipment, and this is attributed to the greater reliability associated with a direct pull coupled with a greater sensitivity of load measurement. These results relate to tests on cubes, and comparison of the value of 5·4 % for direct-pull tests on 10 mm aggregate with the average value of 8·6 % obtained for similar concrete at various levels within the beams indicates the increased variability to be expected when testing full-scale structural members. Each beam was cast from seven similar batches under laboratory conditions, and it is unlikely that in situ concrete will demonstrate lower variability in practice. The beam results also clearly indicate the variations in quality to be expected within members, which must be considered in the planning and interpretation of any in situ strength assessment.

25. The tests to determine the effect of stress suggest that no significant change can be attributed to this cause, although there is an indication of increased scatter of results. These findings do not agree with earlier suggestions,[10] and it appears that more extensive tests are required before firm conclusions can be reached on this. It is clear though that tests should not be taken at points where visible cracks pass through the failure zone.

26. One of the major advantages of this test in comparison with other non-destructive approaches is that a direct measurement is made of a strength property, thus reducing the need to correlate strength with some other property, with the corresponding loss of reliability. It is to be anticipated therefore that other variables influencing the correlation between measured force, or torque, and compressive strength will be few. This is confirmed by the close agreement of the varying age calibration curves in Figs 6 and 7 obtained for the differing aggregate properties, and that derived from Fig. 8, which was obtained by aggregate/cement and water/cement variations, although in the latter case a greater scatter of results is observed, as may be reasonably expected.

27. The test relates only to a surface zone, and it is possible that carbonation effects may affect readings on old concrete. It is likely that the depth of initial failure of 17 mm is sufficient to avoid this problem in most practical situations, although tests to confirm this are still in progress.

28. The tests described relate to only one set of standardized values for bolt size and depth, and reaction ring diameter, and it may well be that these are not the optimum values. An increase in depth, although desirable to reduce localized aggregate effects, may cause problems of interference with reinforcement and increase in load. This would in turn require a larger bolt and heavier loading equipment with reduced sensitivity. Paterson[18] has examined the load behaviour of larger fixings in concrete in detail, and his results suggest similar bolt pull-out characteristics to those reported here. It is thus unlikely that any major advantage is to be gained by changing these proposed standardized values.

29. Despite the small influence of mix characteristics, the high observed scatter of measured values may be attributed to two principal causes: hole drilling and preparation, and the effect of aggregate particles on load transfer and failure.

30. Care must be taken to produce a hole of the correct size which is normal to the concrete surface. This is particularly difficult when drilling concretes of high strength or with large aggregate particles. The tendency is to produce an oversize hole and this, coupled with incomplete dust removal, may cause slippage of the bolt.

31. The nature of the load transfer is such that the location of aggregate particles relative to the wedge may be expected to have a considerable influence on the load required to cause cracking of the matrix. This lack of homogeneity is obviously greater with increasing aggregate size, and the effect of this on result variability has been demonstrated. Preliminary tests on artificial lightweight aggregate concrete also show that a significantly different relationship exists for such materials.

32. The results in Figs 6 and 7 suggest that for laboratory cube specimens a strength estimate based on the mean of four pull-out tests can be expected to have 95 % confidence limits of the order of ± 20 % for 10 mm aggregate and ± 30 % for 20 mm aggregate at the 25 N/mm^2 cube strength level using the torquemeter method. These results relate to specific mixes at varying ages, and it is to be anticipated that the accuracy would be improved by the use of the Author's equipment. This is not directly indicated by Fig. 8, as this relates to a wide variety of mixes which in turn increases the scatter of results. These accuracies for the torquemeter approach may be regarded as applicable to ideal conditions; in practice tests will be required on structural members, possibly cast in situ, and it has been shown that under such conditions testing variability increases.

33. Under comparable ideal conditions, strength prediction accuracies of ± 15 % may be possible to rebound tests[19] and ± 20 % for ultrasonic pulse velocities,[20] but both of these require a specific calibration for the particular mix. In situations where specific charts for the concrete mix cannot be obtained, Fig. 8 suggests that a strength prediction of ± 20 % can be achieved for 10 mm mixes using the Author's equipment, which is considerably better than can be achieved by either of the other approaches in such circumstances. This value is also comparable with that possible with small diameter cores,[21] although it is significantly worse than for cores of 100 mm diameter or greater. Pull-out testing can thus offer an alternative to small cores of comparable accuracy, with considerable savings of time, expense and disruption, for situations in which the strength of unknown concrete is required and where cores of 100 mm or greater are not feasible.

Conclusions

34. It has been suggested that pull-out testing should be regarded solely as a method of comparing surface concrete qualities. The use of cast-in inserts for quality control and specification compliance purposes will essentially be an application of this nature; however, the expanding wedge-anchor bolt approach is unlikely to be of great value in this respect owing to the anticipated scatter of results. In such comparative situations, where specific calibration charts can be obtained easily, it is likely that either surface hardness or ultrasonic pulse velocity testing will prove to be more convenient and of comparable or greater

reliability. The chief advantage of this pull-out approach thus appears to lie in the ability to use a general strength calibration chart relating only to the loading method and possibly aggregate size. This may be of particular value in situations when a strength estimate is required for in situ concrete of unknown age and composition, especially if only one surface is exposed. In such situations, the following conclusions drawn from the experimental results should be considered during the planning of the investigation and interpretation of results.

35. Although the scatter of results is influenced by aggregate size, the effect of aggregate type and size on the relationship between pull-out test results and actual concrete strength is small for granites and gravels of up to 20 mm maximum size, and this may be expected to hold true for other types of natural aggregate. The need for specific calibration for individual mixes is thus reduced, offering considerable advantages over other non-destructive methods when dealing with an unknown concrete. The use of this method for aggregates larger than 20 mm is not recommended because of the limited test depth.

36. Results have been found to vary considerably according to load application and measurement technique, and it is essential that calibration charts are appropriate to the method used.

37. The apparatus developed by the Author, which provides a direct mechanically controlled pull to the bolt, offers advantages of increased sensitivity and reduced test variability when compared with the torquemeter approach.

38. Flexural stresses present in the member under test have no apparent influence on the test results, other than a possible increase in variability.

39. Pull-out tests should not be made where visible cracks pass through the test area.

40. Based on the mean of four pull-out results, 95 % confidence limits of strength prediction are unlikely to be better than ± 20 % for 10 mm aggregates and ± 30% for 20 mm aggregates using the torquemeter under laboratory conditions with specific mix calibrations. These limits are likely to be wider for tests on full-scale structural members because of strength variation within members.

41. Use of the Author's apparatus is likely to increase the accuracy of strength predictions. For 10 mm aggregates it is suggested that 95 % confidence limits of ± 20 % can be achieved by the mean of four results in conjunction with a generalized calibration curve.

42. Pull-out testing using an expanding sleeve wedge-anchor bolt in a drilled hole offers a method of estimating the strength of concrete of comparable accuracy with that achieved by small diameter cores, but with considerable savings of time, expense and disruption.

Acknowledgements

43. The Author is grateful to Miss S. Herriman, Mr S. Sindaha and Mr F. A. Salman for their assistance in obtaining experimental results on which this Paper is partly based.

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Proc. Instn Civ. Engrs, Part 2, 1982, **73**, Mar., 249–252

8442 DISCUSSION

Concrete strength determination by pull-out tests on wedge-anchor bolts

J. H. BUNGEY

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The use of pull-out tests for the determination of concrete strength is controversial as it is uncertain to what extent it is the compressive strength which is measured. Analysis of the pull-out by the theory of plasticity suggests that this question is related to the shape of the failure, and that a direct relationship between pull-out force and concrete compressive strength is obtained if—and only if—the failure surface is constrained to be conical with a half angle approximately equal to the angle of friction for the concrete.

45. In the test investigated in the Paper, the diameter of the reaction ring is 80 mm, and the average failure depth is reported to be 17 mm. Thus the half angle of the failure is 78°, i.e. substantially larger than the angle of friction, which is approximately 37°. If the modified Coulomb failure criterion for concrete is adopted, the failure is predicted to be a combination of sliding and separation, which means that the pull-out force will depend on the tensile strength of the concrete. This might explain the curvature of the calibration curve and the scatter of the results.

46. However, in the Lok test,[3] the failure cone half angle is only 31° (cf. Fig. 1). Consequently, failure takes place by pure sliding, and the pull-out force depends on the compressive strength only, which might explain the low scatter observed.

47. Thus it would seem that a theoretical analysis of the failure provides some insight into the phenomenon, which may be of advantage in the choice of test apparatus. However, the Lok test equipment was designed by trial and error,[22] the plastic analysis being a later development.[23]

48. Assuming the concrete to be rigid, perfectly plastic with the modified Coulomb criterion as the yield condition, and the associated flow rule, the pull-out force F may be determined as a function of imbedment depth h, bolt diameter d and support diameter D.[24,25] With $d/h = 6/17 = 0{\cdot}35$, $D/h = 80/17 = 4{\cdot}7$ and neglecting the tensile strength f_t

$$F = 0{\cdot}125\pi(d + h)hf_{cyl}$$

f_{cyl} being the cylinder strength. Assuming the cube strength to be $f_c = 1{\cdot}25f_{cyl}$, this means that the BRE test would give for $f_t = 0$

$$F/f_c = 123 \text{ mm}^2$$

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As expected, this underestimates the pull-out forces observed (cf. Fig. 8).

49. The assumption of an effective tensile strength equal to 1% of the cylinder strength changes the relationship to

$$F/f_c = 157 \text{ mm}^2$$

for $f_t = f_{cyl}/100$. This agrees quite well with the test points in Fig. 8 corresponding to the highest strengths (F = 7·85 kN for f_c = 50 N/mm^2).

50. In this case the predicted failure surface does not reach the reaction ring. Increase of the assumed tensile strength results in further contraction of the predicted failure, and for a realistic tensile strength level of 10% of the cylinder strength it becomes almost conical, with a half angle equal to the angle of friction, assumed at 37°. In that case the predicted relationship is

$$F/f_c = 200 \text{ mm}^2$$

for $f_t = f_{cyl}/10$. This corresponds to the lowest points in Fig. 8 (F = 4·0 kN for f_c = 20 N/mm^2).

51. As the relative tensile strength f_t/f_{cyl} decreases with increasing strength level, the calibration curve is expected to be curved, as borne out by Fig. 8, although the plastic analysis considerably overestimates the effect.

Dr A. J. Chabowski and Mr D. W. Bryden-Smith,
Building Research Establishment

Our original paper[10] was based only on the comparatively few results which were available at the time. Subsequently[26,27] we have presented the findings of more extensive investigations which are now the basis for use of the technique by most testing houses; the necessary equipment is available commercially.

53. Our general calibration curve now includes concretes made with crushed rock or gravel aggregates up to 20 mm maximum size and gives a 95% lower confidence limit 22% below the mean strength.[26,27] This has been achieved by rejecting any set of test results from a cube where at least four results of the six tests made on each cube had a coefficient of variation greater than 16%. On site, a set of four torque values having a similar variability (or falling within the upper and lower confidence limits for torque) must be obtained.[27] The differences in calibration reported in the Paper can perhaps be attributed to the use of different procedures from those given in reference 10; the mean coefficient of variation of 16·5% for the 20 mm aggregate concretes is surprisingly high (Table 5) suggesting that more tests should have been made on about half of the cubes and the results should have then been either rejected or repeated.

54. The effect of precompression in increasing the scatter of results referred to in the Paper is automatically allowed for by the selection procedure just described.[26] Dr Ash, under contract to the BRE, investigated the effect which resulted in our advice that precompression exceeding 4 N/mm^2 should be allowed for.[27]

55. In relation to the influence of loading technique the use of a fully ratchetting torque wrench for loading has several advantages. It is light and compact, it can be used in corners and near to obstructions and it has a maximum torque indicator. The so-called settling pauses have a dual function: to check the rate of increase of force (torque) to confirm that the test is proceeding satisfactorily and to ensure smoothness of loading.

56. Other methods of estimating the quality of concrete in situ, such as ultrasonic pulse velocity and rebound hammer, have their place in the assessment of concrete strength but require an initial determination of strength to make the results meaningful. The internal fracture test can provide this first step, with little damage as compared with coring, and being independent of the type of concrete and size of aggregate up to 20 mm maximum size can be used as a calibration point for such methods.

Mr Bungey

Dr Braestrup has indicated the extensive work that has been undertaken to apply plasticity theory to problems of this type. This confirms the major advantage of the Lok test configuration which gives results related only to compressive strength rather than a combination of sliding and tension. As Dr Braestrup indicates, the geometry of the internal fracture test gives a failure cone half angle which is considerably greater than the angle of friction of the concrete and results in a more complex failure mode. The internal fracture test has developed from the proposals of Dr Chabowski and Mr Bryden-Smith and the test depth appears to have been based on a number of practical factors, such as likely cover to reinforcement and load capacity of readily available expanding anchor bolts. The reasons for the choice of reaction tripod diameter are unclear but in the light of Dr Braestrup's remarks it may be that more reliable results could be obtained if this were reduced. It is not practicable to adopt the diameter of 26 mm which would be necessary to produce a cone half angle of 37°.

58. Dr Chabowski and Mr Bryden-Smith have indicated a procedure in which the range of results is artificially constrained, this being necessary because of the high scatter obtained with the torquemeter approach. It is true that the coefficient of variation of 16·5% given in Table 5 could probably be reduced if more readings were taken. However, it is practically desirable that damage is limited by keeping the number of test points to a minimum. Reduced testing scatter is the principal advantage of the direct pull method described and, although the equipment is slightly more cumbersome than the torquemeter, operation is no more complicated.

59. With respect to the suggestion that calibration discrepancies indicated in Figs 6 and 7 may be attributed to differences of procedure, it can only be stated that their recommended methods[10] were followed under laboratory conditions. It seems likely that on site even greater differences may occur. Long and Glass[28] have suggested similar overoptimism of strength estimates using Chabowski and Bryden-Smith's calibration. The most recent version of their calibration[26] is $F = 3{\cdot}116T^{1{\cdot}69}$ based on 150 mm cubes, and this makes a small difference to the earlier curve shown in Figs 6 and 7 which has been corrected to allow for the cube size effect. This change further increases the discrepancy, especially at higher strengths.

60. There can be no doubt that the internal fracture test is a valuable guide to strength assessment if used and interpreted properly. However, if, as with some other non-destructive methods, procedures are oversimplified, the need for skilled application can be overlooked. If the confidence of engineers in this form of testing is to be developed, special care is necessary to ensure that the approach is not misused.

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Paper 15

Contribution to Discussion:

"Assessing the strength of insitu Portland Cement Concrete by Internal Fracture Tests"

(A. J. Chabowski and D. W. Bryden-Smith)

Mag. of Conc. Res Vol. 33 No. 116 September 1981
pp.187-188

Assessing the strength of in situ Portland cement concrete by internal fracture tests*

A. J. Chabowski and D. W. Bryden-Smith

Contribution by J. H. Bungey
University of Liverpool : Department of Civil Engineering

Dr Chabowski and Mr Bryden-Smith have produced valuable data confirming and enhancing the value of the test approach they discuss. The significant feature of this method is that, if it is properly executed, an estimate of in situ concrete strength can be obtained from a single generalized calibration curve for natural aggregates. Although limited to the surface zone, this is something that cannot justifiably be claimed for any other non-destructive or semi-destructive method. The principal application is therefore most likely to be as an alternative to cores in situations where these cannot easily be obtained, although the accuracy of strength prediction is inferior even to that of small-diameter cores[1].

A recently published account[2] of work in this area has demonstrated that a mechanical direct-pull loading method, a version of which is shown in Figure I, offers greater sensitivity of load application and measurement than the torque meter. The resulting reduction of test variability means that an improved accuracy of strength prediction is possible and has enabled a clear distinction in test variability to be established between 10 and 20 mm aggregate concrete. Furthermore the importance of careful control of load application rate must not be overlooked, and techniques involving 'settling pauses' were found to yield less consistent results than steady loading application. It is particularly important that a calibration appropriate to the loading method is used; this cannot be over-emphasized, since the authors' curves have been observed in use with a totally different hydraulic loading method by a commercial organization carrying out testing in situ.

I agree with Dr Chabowski and Mr Bryden-Smith that the effect of compressive stress upon the magnitude of measured pull-out force is small. Tests on 4·8 m × 500 mm deep beams under load suggest, however, that the coefficient of variation of test results will show a marked increase under even low levels of either compressive or tensile stress. This is

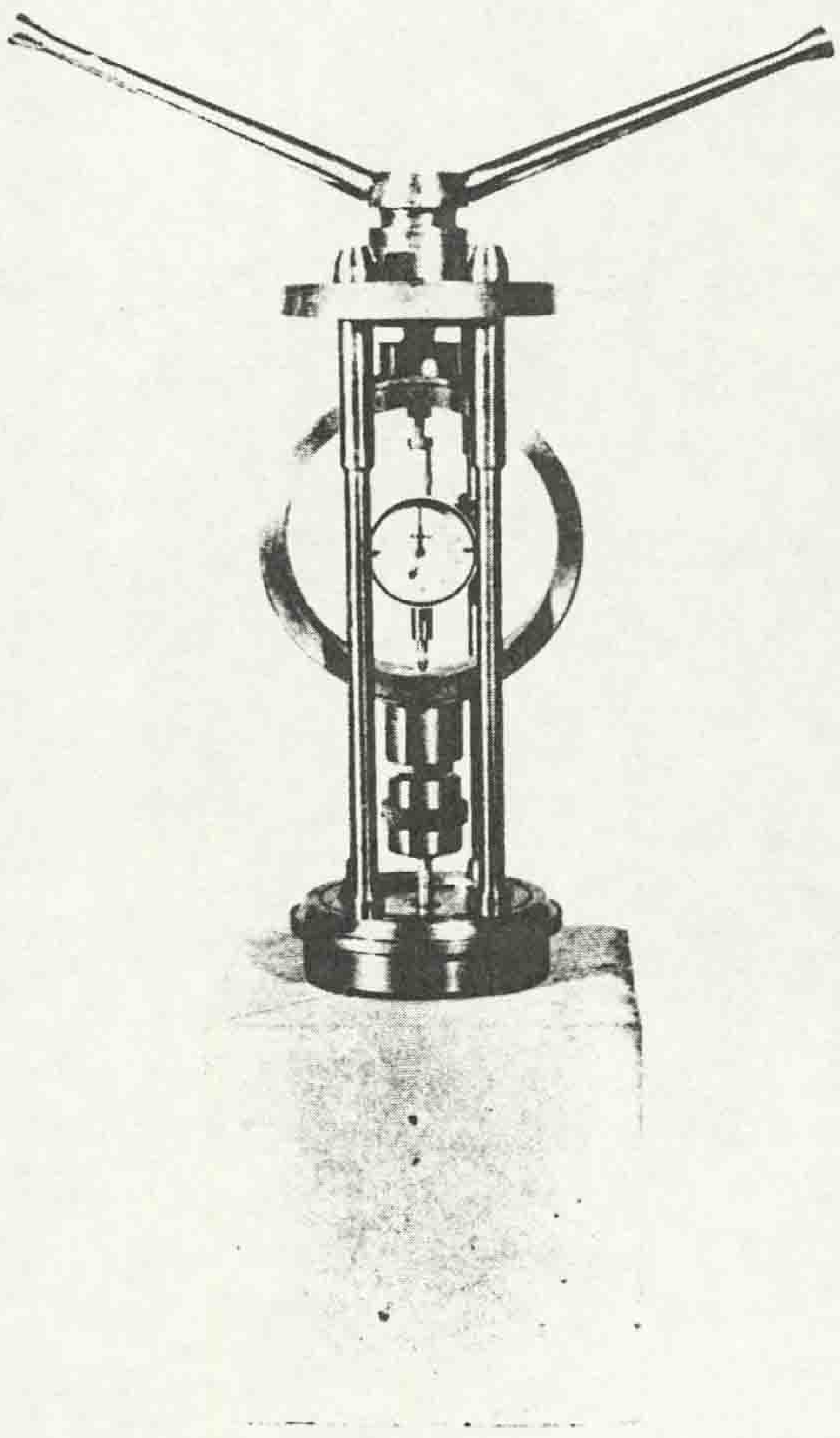

Figure I: Equipment for mechanical direct-pull tests.

*Pages 164–172 of *MCR* 112.

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TABLE 1: **Variability of pull-out tests using direct-pull equipment.**

Average coefficient of variation (%)		
Cubes	Unloaded beams	Loaded beams
5·4	8·6	12·2

indicated in Table I, which relates to 10 mm North Notts gravel concrete in cubes and at individual levels of beams, which were tested unloaded and again when under load. Tests on cubes of similar concrete using torque meter equipment yielded an average coefficient of variation of 12·4%, and it would thus be interesting if the authors could provide information concerning the variability of the measured torque values of their tests on both cubes and stressed specimens. This may be of importance in the interpretation of the results of in situ tests, since their quoted accuracies of strength prediction appear to be based on the results of tests on cubes. It may be that the number of tests necessary to achieve comparable accuracies should be increased for members tested in service.

Reply by authors

We thank Mr Bungey for his contribution and would like to reply as follows.

The internal fracture test equipment was developed for ease of application in restricted areas on horizontal surfaces on top of and underneath concrete elements and on vertical surfaces of columns. Development work on the test included use of equipment with a load-cell, hydraulic jack, continuous load application, different sizes of reaction ring, different depths of anchorages and different makes and sizes of anchor bolt.

Preliminary tests suggested that the bolt size, the depth of anchor and the size of reaction ring should be standardized. For simplicity and practical use on site, load-cell and hydraulic jack were not adopted and the method of operation was standardized to include 'settling pauses' to avoid any timing requirement in steady continuous load application.

The maximum coefficient of variation in all tests was 16%; the average, obviously a lower figure, was not calculated. Small differences in variability between 10 and 20 mm maximum aggregate size concrete could be distinguished, but to make the method general and applicable to concrete mixes of unknown composition (as found on practical sites) the differences were disregarded.

Differences also existed between HAC concrete, and crushed-rock and gravel-aggregate concretes, and separate calibration curves were shown in the subsequent BRE Information Paper IP 22/80 but, because the effect was relatively small, the average relationship was recommended for practical use.

The effect of compressive stress upon the results can be ignored provided that locations for test are chosen where compressive stresses are low – say, less than 4 N/mm² – otherwise a correction of 1% of estimated cube strength for each 1 N/mm² of compression is suggested. Higher precompression cannot be ignored but, since the scatter of results found in the experimental work was considerable, the results were shown in full in the paper to allow engineers to make their own decisions.

The graphs were based on tests on cubes, because cubes provided a means of testing the same concrete both to failure in compression and in the internal fracture test, thus eliminating the difference in strength between reference cubes and the specimens. This difference can be substantial, especially with crushed-rock aggregates and sands. It is considered that the number of tests recommended is sufficient to assess the compressive strength of structural members in situ. It is open to the engineer concerned to consider the use of more tests; however, we suggest that the law of diminishing returns will be operating and the cost of additional tests is likely to outweigh the increase in accuracy of the mean of sampling tests. The accuracy of prediction of compressive strength of concrete depends upon the scatter of points around the calibration curve, i.e. the width of the confidence limit band.

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Paper 16

"Planning and Interpretation of
Insitu Testing"

The Testing of Concrete in Structures
Surrey University Press 1982
Chapter 1

1 Planning and interpretation of in-situ testing

A great deal of time, effort and expense can be wasted on in-situ testing unless the aims of the investigation are clearly established at the outset. These will affect the choice of test method, the extent and location of the tests and the way in which the results are handled—inappropriate or misleading test results are often obtained as a result of a genuine lack of knowledge or understanding of the procedures involved. If future disputes over results are to be avoided, liaison of all parties involved is also essential at an early stage in the formulation of a test programme. Engineering judgement is inevitably required when interpreting results, but the uncertainties can often be minimized by careful planning of the test programme.

1.1 Aims of in-situ testing

Three basic categories of concrete testing may be identified.

(a) *Control testing* is normally carried out by the contractor or concrete producer to indicate adjustments necessary to ensure an acceptable supplied material.

(b) *Compliance testing* is performed by the engineer on the concrete delivered according to an agreed plan, to judge compliance with the specification.

(c) *Secondary testing* is carried out on hardened concrete either in, or extracted from, the structure. This may be required in situations when control and compliance results are unavailable or inappropriate, as in an old or deteriorating structure, or when there is doubt about their reliability.

Control and compliance tests are generally performed on "standard" hardened specimens made from samples of the same concrete as used in a structure; it is less common to test fresh concrete. There are also some instances in which in-situ tests on the hardened concrete may be used for this purpose. This is most common in the precasting industry for checking the quality of standardized units, and the results can be used to monitor the uniformity of units produced as well as their relationship to some pre-established minimum acceptable value. There is, however, an increasing awareness amongst engineers that "standard" control specimens, although notionally of the same material, may misrepresent the true quality of concrete actually in a structure. This is due to a variety of causes including non-uniform supply of material and differences of compaction, curing and general workmanship. As a result, a trend towards in-situ compliance testing, using methods which are either non-destructive or cause only very limited damage, is emerging, particularly in North America. Such tests are likely to be used as a back-up for conventional control testing. They offer the advantage of early warning of suspect strength, coupled with the detection of defects which may otherwise be unknown but lead to long-term durability problems.

The principal applications of in-situ tests will be as secondary testing, which may be necessary for a wide variety of reasons. These fall into two basic categories.

1.1.1 *Compliance with specification*

The most common example of this is where additional evidence is required in contractual disputes following non-compliance of standard specimens. Other instances involve retrospective checking following deterioration of the structure, and will generally then be related to apportionment of blame in legal actions. Strength requirements form the basis of most specifications, and the engineer must select the most appropriate methods of assessing the in-situ strength on a representative basis, with full knowledge of the likely variations to be expected within various structural members (as discussed in section 1.4). The results should be interpreted to determine in-situ variability as well as strength, but a major difficulty arises in relating measured in-situ strength to anticipated corresponding control specimen strength at a specific but different age. This problem is examined in detail in section 1.4.2.

Minimum cement content will often be specified, since durability is a major requirement, and chemical tests may be necessary to confirm compliance. Chemical tests may also be required to check for the presence of forbidden admixtures, entrained air, and cement content where deterioration has occurred. Poor workmanship is often the principal cause of durability problems, and tests may also be aimed at demonstrating inadequate cover or compaction, incorrect reinforcement quantities or location, or poor quality of specialist processes such as grouting of post-tensioned construction.

1.1.2 *Assessment of in-situ quality*

This is primarily concerned with the adequacy of the existing structure, and it is important to distinguish between the need to assess the properties of the

material, and the performance of a structural member as a whole. The need for testing may arise from a variety of causes which include

(a) proposed change of usage or extension of a structure;
(b) acceptability of a structure for purchase or insurance;
(c) assessment of structural integrity or safety following material deterioration, or structural damage such as caused by fire, blast or overload;
(d) serviceability or adequacy of members known or suspected to contain material which does not meet specifications, or with design faults;
(e) Monitoring of strength development in relation to formwork stripping, prestressing or load application.

Although in specialized structures, features such as density or permeability may be relevant, generally it is the in-situ strength that is regarded as the most important criterion. For strength monitoring during construction, it will normally only be necessary to compare test results with limits established by trials at the start of the contract, but in other situations a prediction of actual concrete strength is required to incorporate into calculations of member strength. One exception may be insurance or purchase assessments, in which case visual inspection may be adequate unless there are signs of distress, or doubt exists about the structure. Where calculations are to be based on measured in-situ strength, careful attention must be paid to the location of tests and the validity of the safety factors adopted, and this problem is discussed in section 1.5.

Difficulties in obtaining an accurate quantitative estimate of in-situ concrete strength can be considerable; wherever possible the aim of testing should be to compare suspect concrete with similar concrete in other parts of the structure which is known to be satisfactory, or of proven strength.

Investigation of the overall structural performance of a member is frequently the principal aim of in-situ testing, and it should be recognized that in many situations this would be most convincingly demonstrated directly by means of a load test. The confidence attached to the findings of the investigation may then be considerably greater than if member strength predictions are derived indirectly from strength estimates based on in-situ materials tests. Load testing may however be prohibitively expensive or simply not a practical proposition.

1.2 Test methods available

Details of particular methods are given in subsequent chapters; these may be classified in a variety of ways. Table 1.1 lists the principal tests in terms of the

Table 1.1 Tests on in-situ concrete

Information required	*Methods available*
Member behaviour and strength	Load test with deflection and strain measurements
Concrete strength	Cores Rebound hammer Pull-out and internal fracture Break-off and pull-off Penetration resistance Ultrasonic pulse velocity
Cracking	Ultrasonic pulse velocity Acoustic emission and holography
Honeycombing and compaction	Ultrasonic pulse velocity γ-radiography Cores Pulse echo techniques
Density	γ-radiometry
Permeability	Absorption, flow tests and capillary rise
Moisture content	Nuclear methods Electrical resistivity Microwave absorption
Cement content	Chemical analysis Nuclear methods
Mix properties and constituents	Chemical analysis Cores Micrometric methods
Reinforcement detection	Magnetic methods X- and γ-radiography
Concrete deterioration	Chemical analysis Thermoluminescence Ultrasonic pulse velocity Micrometric methods
Abrasion resistance and soundness	Rebound hammer Wear tests Physical methods Infrared thermography

information required, but where a number of options are available, considerations of damage, cost, time and reliability will be important.

Estimation of concrete or member strength is the most common requirement of in-situ investigations, but unfortunately none of the available methods can be used to provide a reliable value in every situation. Table 1.2 indicates the likely damage resulting from such tests, together with the principal practical restrictions in each case (damage to finishes is almost inevitable, whatever method is used). Table 1.3 classifies these same tests in terms of their relative cost, speed and reliability for strength determination.

Other test methods given in Table 1.1 will generally be non-destructive,

Table 1.2 Strength tests—damage and restrictions

Test method	*Probable damage*	*Major restrictions*
Collapse load test	Member destroyed	Member must be isolated, and preferably removed, from rest of structure prior to test
Overload test	Possible loss of member	Member must be isolated, or allowance made for load distribution to adjacent parts of the structure Extensive safety precautions to cater for possible collapse
Cores	Holes to be made good	Limitations of core size and proportions Safety precautions for critical members
Penetration resistance (Windsor probe)	Cone approx. 50 mm dia. to be made good	Minimum edge distance Minimum member thickness
Pull-out	Bolt hole remains	Preplanned
Internal fracture	Bolt to be cropped or cone approx. 75 mm dia. to be made good	Drilling difficulties
Ultrasonics	None	Two smooth surfaces necessary
Rebound hammer	None (if concrete mature)	Smooth surface necessary

except for chemical and petrographic methods which require small samples to be cut or drilled from the concrete. Many of these other methods are of a highly specialized nature requiring expensive equipment, with extensive safety precautions in some cases. Their cost is therefore likely to be high with limited benefit, and their use restricted to cases where no alternative exists. These aspects, together with relative reliabilities, are discussed in the sections of this book dealing with the various individual test methods.

Table 1.3 Strength tests—relative merits

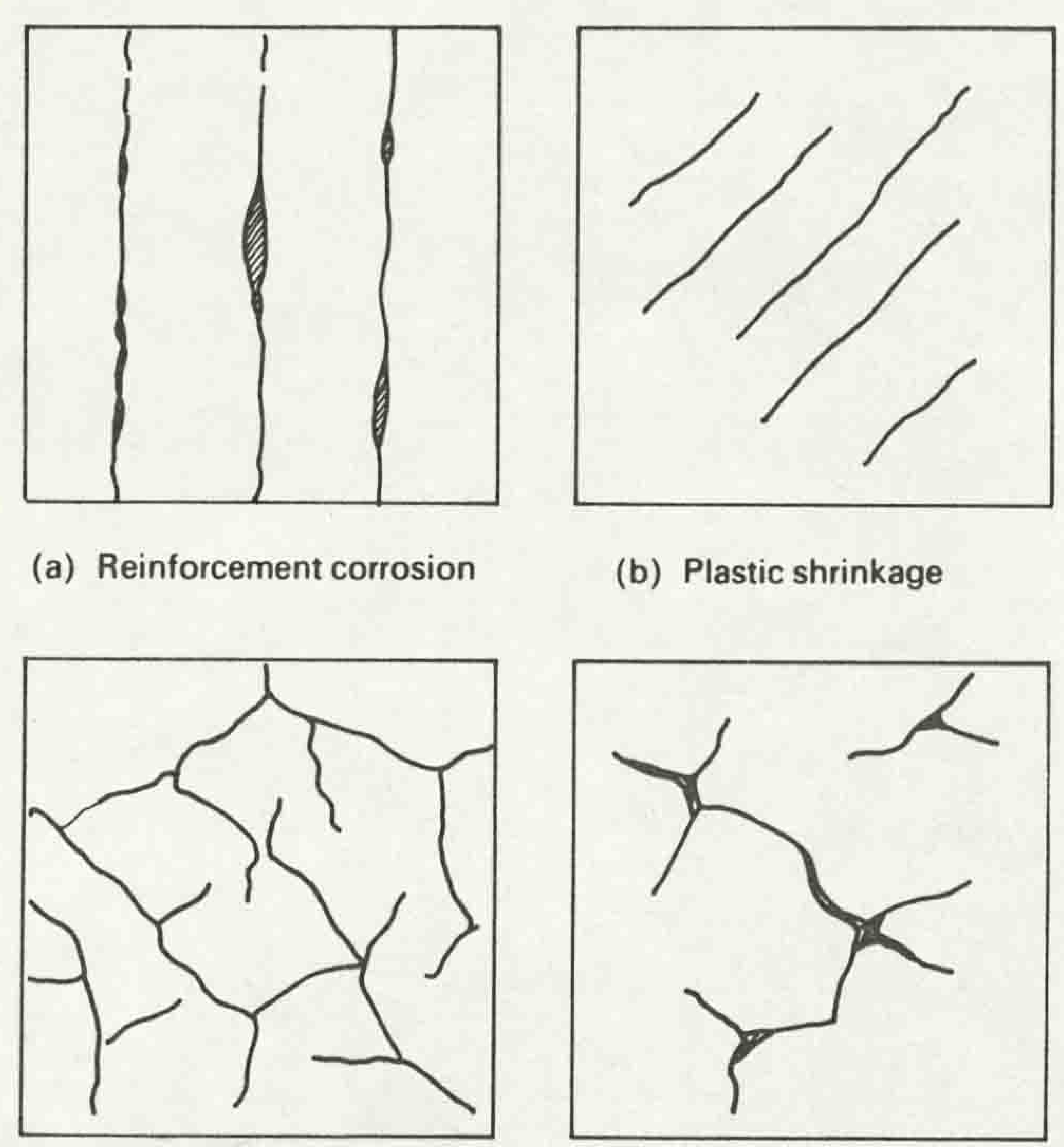
Test method	*Cost*	*Speed of test*	*Damage to concrete*	*Representativeness*	*Reliability of strength calibrations*
Collapse load test	High	Slow	Total	Good	Good
Overload test	High	Slow	Variable	Good	Good
Cores	High	Slow	Moderate	Moderate	Good
Penetration resistance	Moderate	Fast	Minor	Near surface only	Moderate
Pull-out/ Internal fracture	Moderate	Fast	Minor	Near surface only	Moderate
Ultrasonics	Low	Fast	None	Good	Moderate
Rebound hammer	Very low	Fast	Unlikely	Surface only	Poor

1.3 Test programme planning

This involves consideration of the most appropriate tests to meet the established aims of the investigation, the extent or number of tests required to reflect the true state of the concrete, and the location of these tests. Visual inspection is an essential feature whatever the aims of the test programme, and will enable the most worthwhile application of the physical tests which have been summarized in section 1.2. A number of typical illustrative examples of test programmes to meet specific requirements are given in Appendix A.

1.3.1 *Visual inspection*

This can often provide valuable information to the well-trained eye. Visual features may be related to workmanship, structural serviceability and material deterioration, and it is particularly important that the engineer be



(a) Reinforcement corrosion (b) Plastic shrinkage

(c) Sulphate attack (d) Alkali/aggregate reaction

Figure 1.1 Typical crack types.

able to differentiate between the various types of cracking which may be encountered. Figure 1.1 illustrates a number of these in their typical forms.

Segregation or excessive bleeding at shutter joints may reflect problems with the concrete mix, as might plastic shrinkage cracking, whilst honeycombing may be an indication of low standards of construction workmanship. Lack of structural adequacy may show itself by excessive deflection or flexural cracking, and this may frequently be the reason for an in-situ assessment of a structure. Long-term creep deflections, thermal movements, or structural movements may cause distortion of door frames, cracking of windows, or cracking of a structure or its finishes. Visual comparison of similar members is particularly valuable as a preliminary to testing to determine the extent of the problem in such cases.

Material deterioration is often indicated by surface cracking and spalling of the concrete, and examination of crack patterns may provide a preliminary indication of the cause. The most common causes are reinforcement corrosion due to inadequate cover or high chloride concentrations, and concrete disruption due to sulphate attack, frost action or alkali-aggregate reactions. As shown in Figure 1.1, reinforcement corrosion is usually indicated by splitting and spalling along the line of bars, whilst sulphate attack may produce a random pattern accompanied by a white deposit leached on the surface. Alkali-aggregate reaction is generally (but not necessarily) characterized by a star-shaped crack pattern (1), whilst frost attack may give patchy surface spalling and scabbing. Because of similarities it will often be impossible to determine causes by visual inspection alone, but the most appropriate identification tests can be selected on this basis. Pollock, Kay and Fookes (2) suggest that systematic "crack mapping" is a valuable diagnostic exercise when determining the causes of deterioration, and they

Table 1.4 Diagnosis of defects and deterioration

Cause	*Symptoms*			*Age of appearance*	
	Cracking	*Spalling*	*Erosion*	*Early*	*Long-term*
Structural deficiency	×	×		×	×
Reinforcement corrosion	×	×			×
Chemical attack	×	×	×		×
Frost damage	×	×	×	×	×
Fire damage	×	×		×	
Internal reactions	×	×			×
Thermal effects	×	×		×	×
Shrinkage	×			×	×
Creep	×	×			×
Rapid drying	×			×	
Plastic settlement	×			×	
Physical damage	×	×	×	×	×

give detailed guidance about the recognition of crack types. The symptoms relating to the most common sources of deterioration are summarized in Table 1.4, which is based on the suggestions of Higgins (3).

1.3.2 *Test selection*

Test selection will be based on a combination of factors such as non-destructiveness, cost, speed and reliability, and may conveniently follow a procedure such as that shown in Figure 1.2. This example is aimed at assessment of structural safety, but the basic features of visual inspection followed by a sequence of physical tests according to cost, convenience and suitability will generally apply.

In the common situation where an assessment of material strength is required, it is unfortunate that the complexity of calibration tends to be greatest for the test methods which cause the least damage. Whilst surface hardness and pulse velocity tests cause no damage, are cheap and quick, and are ideal for comparative and uniformity assessments, their calibration for absolute strength prediction poses many problems. Core tests provide the most reliable in-situ strength assessment but also cause the most damage and are slow and expensive. They will often be regarded as essential, and their value may be enhanced if they are used to form a basis for calibration of non-destructive or other semi-destructive methods which may then be adopted more widely. Other semi-destructive methods, such as internal fracture and penetration resistance, require less detailed calibration for strength than the non-destructive methods but cause some surface damage, test only the surface zone, and suffer from high variability. The level of accuracy required from the

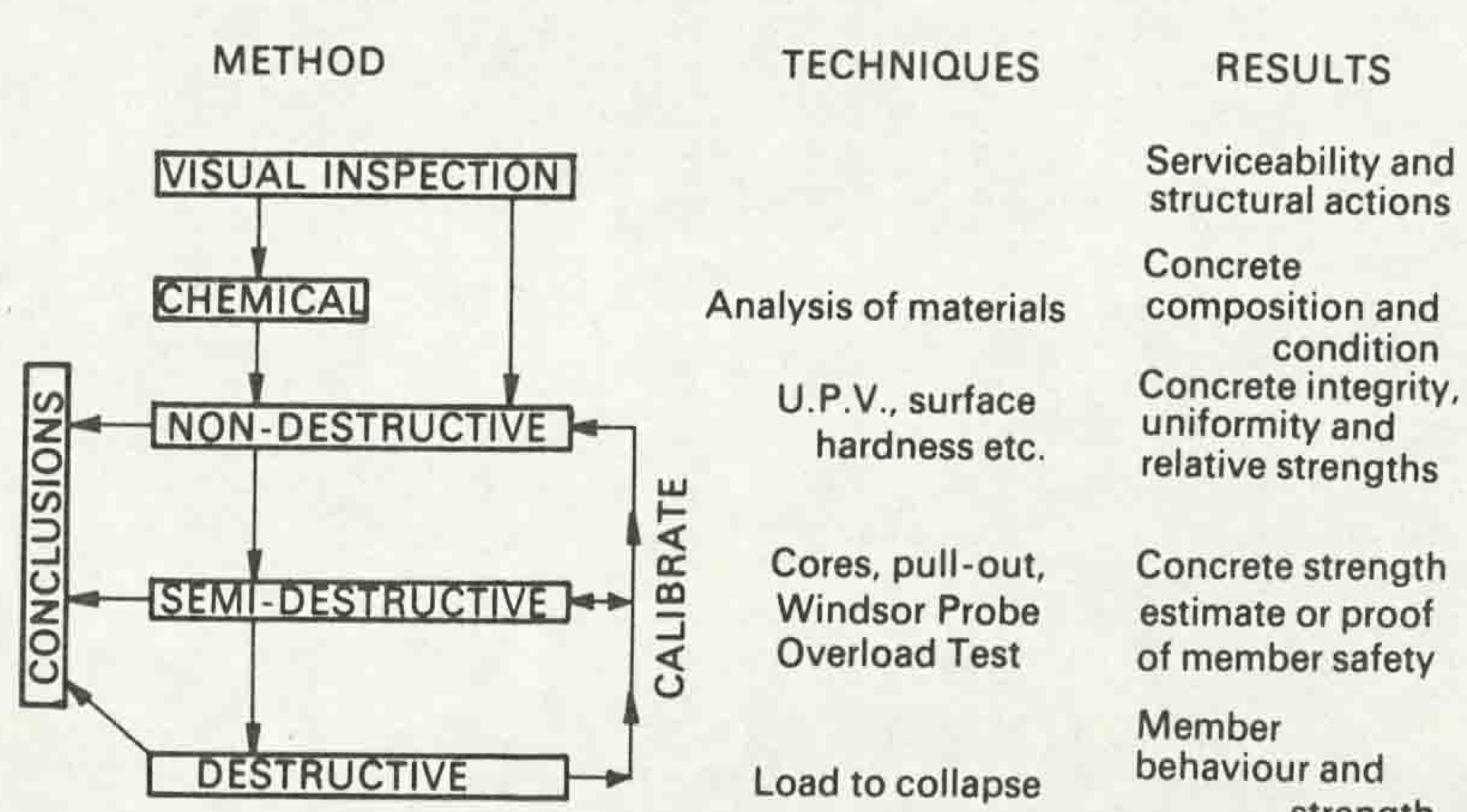


Figure 1.2 Test selection procedures.

strength predictions may be an important factor in selecting the most appropriate methods to use. This must be coupled with the acceptability of making good any damaged areas for appearance and structural integrity.

When comparison with concrete of similar quality is all that is necessary, the choice of test will be dominated by the practical limitations of the various methods. The least destructive suitable method will be used initially, possibly with back-up tests using another method in critical regions. For example, surface hardness methods may be used for new concrete, or ultrasonics where two opposite surfaces are accessible. When there is only one exposed face, penetration resistance testing is quick and suitable for large members such as slabs, whilst pull-out tests may be more suitable for smaller members.

In-situ load tests may be appropriate for parts of a structure where the structural performance is under investigation, but problems of isolating individual members can be substantial. Where a large number of similar elements (such as precast beams) is involved it may be better to remove a few typical elements for laboratory load testing, and to use non-destructive methods to compare these with those remaining in the structure.

The test programme will also be influenced by the costs of the various test methods in relation to the value of the project involved, the costs of delays to construction, and of possible remedial works. Accessibility of the suspect concrete and the handling of test equipment must be considered, together with the safety of site personnel and the general public during testing operations. Typical examples of test programmes suggested for particular situations are included in Appendix A.

1.3.3 *Number and location of tests*

Establishing the most appropriate number of tests is a compromise between accuracy, effort, cost and damage. Table 1.5 lists the number of tests which may be considered equivalent to a single "standard" core. The accuracy of strength prediction will depend in most cases on the reliability of the

Table 1.5 Relative numbers of readings necessary for various test methods

Test method	*No. of individual readings equivalent to a "standard" core*
"Standard" core	1
Small cores	3
Schmidt hammer	10
Ultrasonic pulse velocity	1
Internal fracture	6
Windsor probe	3

calibration used, but for "standard" cores 95% confidence limits may be taken as $\pm(12/\sqrt{n})\%$ where n is the number of cores from the particular location. Where cores are being used to provide a direct indication of strength or as a basis of calibration for other methods, it is important that sufficient are taken to provide an adequate overall accuracy. It is also essential to recognize that the results will relate only to the particular location tested.

For comparative purposes the non-destructive methods are the most efficient since their speed permits a large number of locations to be easily tested. For a survey of concrete within an individual member, at least 40 locations are suggested, spread on a regular grid over the member, whilst for comparison of similar members a smaller number of points on each member, but at comparable positions, should be examined. Where it is necessary to resort to other methods such as internal fracture or Windsor probe tests, practicalities are more likely to restrict the number of locations examined, and the survey may be less comprehensive.

In-situ strength estimates determining structural adequacy should ideally be obtained for critically stressed locations, in the light of anticipated strength distributions within members (described in section 1.4.1). Attention will thus often be concentrated on the upper zones of members, unless particular regions are suspect.

Where specification compliance is being investigated, it is recommended (4) that not less than four cores are taken from the suspect batch of concrete. Where small cores are used, a larger number will be required to give a comparable accuracy, due to greater test variability (5), and probably at least 12 results are required. With other test methods, a minimum number of readings is less clearly defined but should reflect the values given in Table 1.5 coupled with the calibration reliability. It is inevitable that a considerable "grey" or "not proven" area will exist when comparing strength estimates from in-situ testing with specified cube or cylinder strengths, and $\pm 15\%$ has been suggested for a group of four cores (4). This value may increase when dealing with old concrete, due to uncertainties about age effects on strength development. Tests for material specification compliance must be made on typical concrete, and hence the weaker top zones of members should be avoided. Testing at around mid-height is recommended for beams, columns and walls, whilst surface zone tests on slabs must be restricted to soffits unless the top layer is first removed. Care must similarly be taken to discard material from the top 20% (or at least 50 mm) of slabs when testing cores. Tests may also be necessary on areas which show signs of poor compaction or workmanship for comparison with specifications.

In-situ tests for purposes other than strength determination will invariably relate to the location at which the test was made or the sample taken. The number of tests or samples necessary will be a matter of engineering

judgement, but the criteria discussed above for strength testing should provide a useful basis.

The number of load tests that can be undertaken on a structure will be limited, and these should be concentrated on critical or suspect areas. Visual inspection and non-destructive tests may be valuable in locating such regions. Where individual members are to be tested destructively to provide a calibration for non-destructive methods, they should preferably be selected to cover as wide a range of concrete quality as possible.

1.4 In-situ concrete variability

It is well established that the strength of in-situ concrete will vary within a member due to differences of compaction and curing as well as non-uniform supply of material (6, 7). Supply variations will be assumed to be random, but compaction and curing variations follow well-defined patterns according to member type. A detailed appreciation of these variations is essential to planning any in-situ test programme and also to permit sensible interpretation of results.

The average in-situ strength of a member, expressed as the strength of an equivalent cube, will invariably be less than that of a standard cube of the same concrete which has been properly compacted and moist-cured for 28 days. The extent of the difference will depend upon construction techniques, workmanship and exposure, but general patterns can be defined according to member type. This aspect, which is particularly important for interpretation of test results, is discussed in detail in section 1.4.2.

1.4.1 *Within-member variability*

Variations in concrete supply will be due to differences in materials, batching, transport and handling techniques. These will reflect the degree of control over production and will normally be indicated by control and compliance test specimens in which other factors are all standardized. In-situ measurement of these variations is difficult because of the problem of isolating them from compaction and curing effects. They may however be roughly assessed by consideration of the coefficient of variation of tests taken at a number of comparable locations within a member or structure. Compaction and curing effects will depend partially upon construction techniques but are also closely related to member types and location within the member.

Reinforcement may hinder compaction but there will be a tendency for moisture to rise and aggregate to settle during construction. Lower levels of members will further be compacted due to hydrostatic effects, related to member depth, with the result that the general tendency will be for strengths to be highest near the base of pours and lowest in the upper regions. The basic aim of curing is to ensure that sufficient water is present to enable hydration to proceed. For low water:cement ratio mixes, self-desiccation must be avoided by allowing water ingress, whilst for other mixes, drying out must be prevented. Incomplete hydration resulting from poor curing may cause variations of strength between interior and surface zones of members, although a figure of only 5–10% has been suggested for this (7). Differential curing across members may serve to further increase the variations from compactional factors.

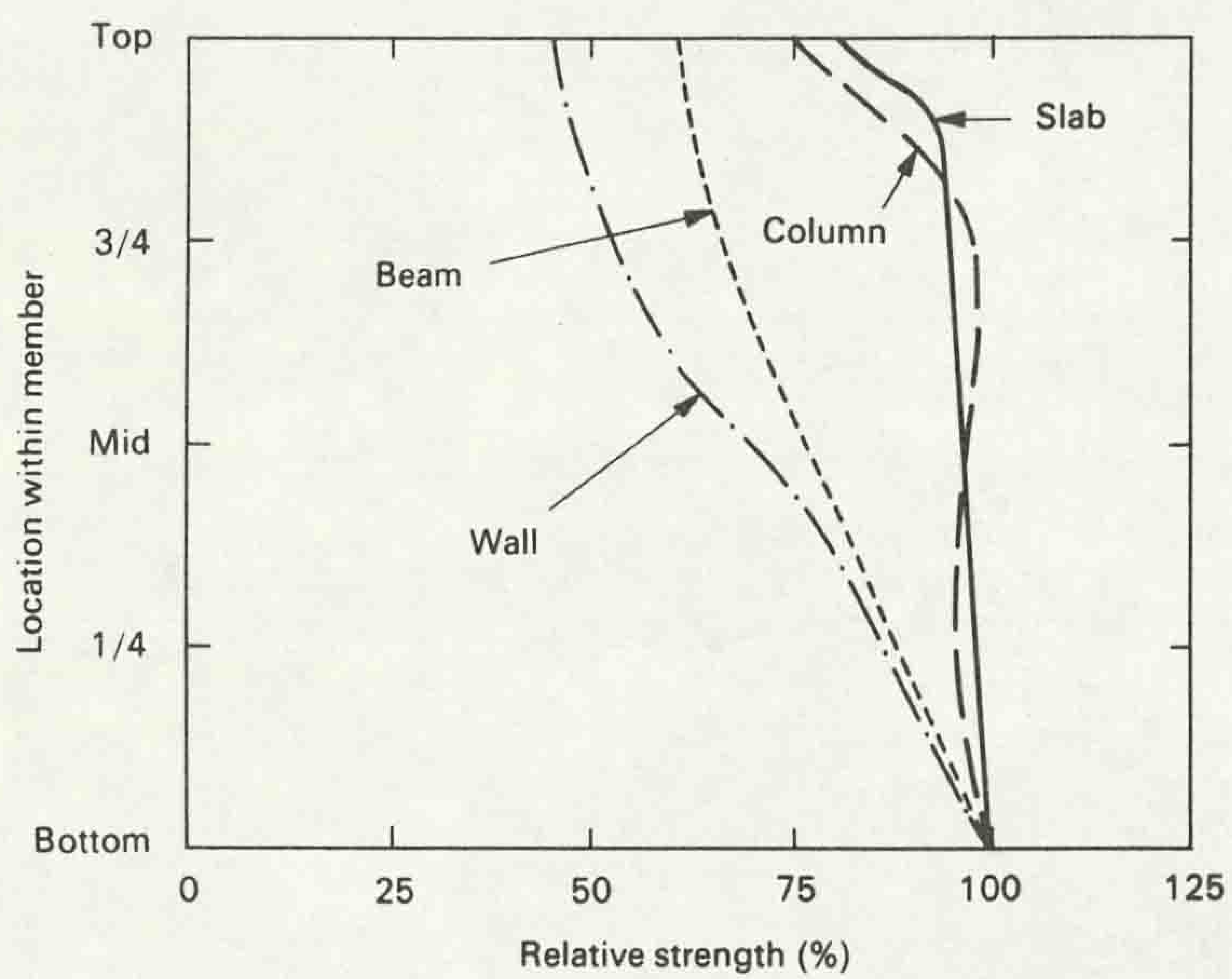


Figure 1.3 Within member variations.

Typical relative strength variations according to member type are illustrated in Figure 1.3. These results have been derived from numerous reports of non-destructive testing including that by Maynard and Davis (8) and can only be regarded as indicating general trends which may be expected, since individual construction circumstances may vary widely. For beams and walls the strength gradients will be reasonably uniform, although variations in compaction and supply may cause the type of variability indicated by the relative strength contours of Figures 1.4 and 1.5. Little data is available for slabs, but it has been suggested that the reduced differential of about 25% across the depths may be concentrated in the top 50 mm (7). Variations in

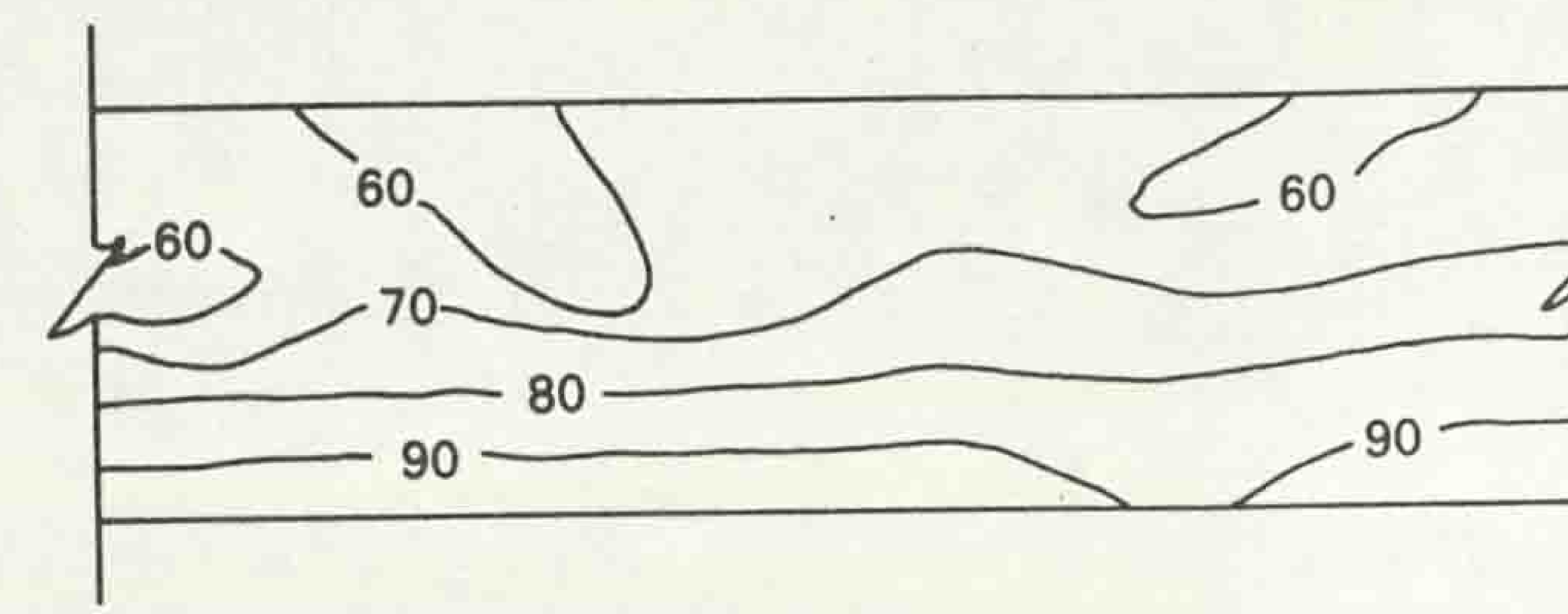


Figure 1.4 Typical relative percentage strength contours for a beam.

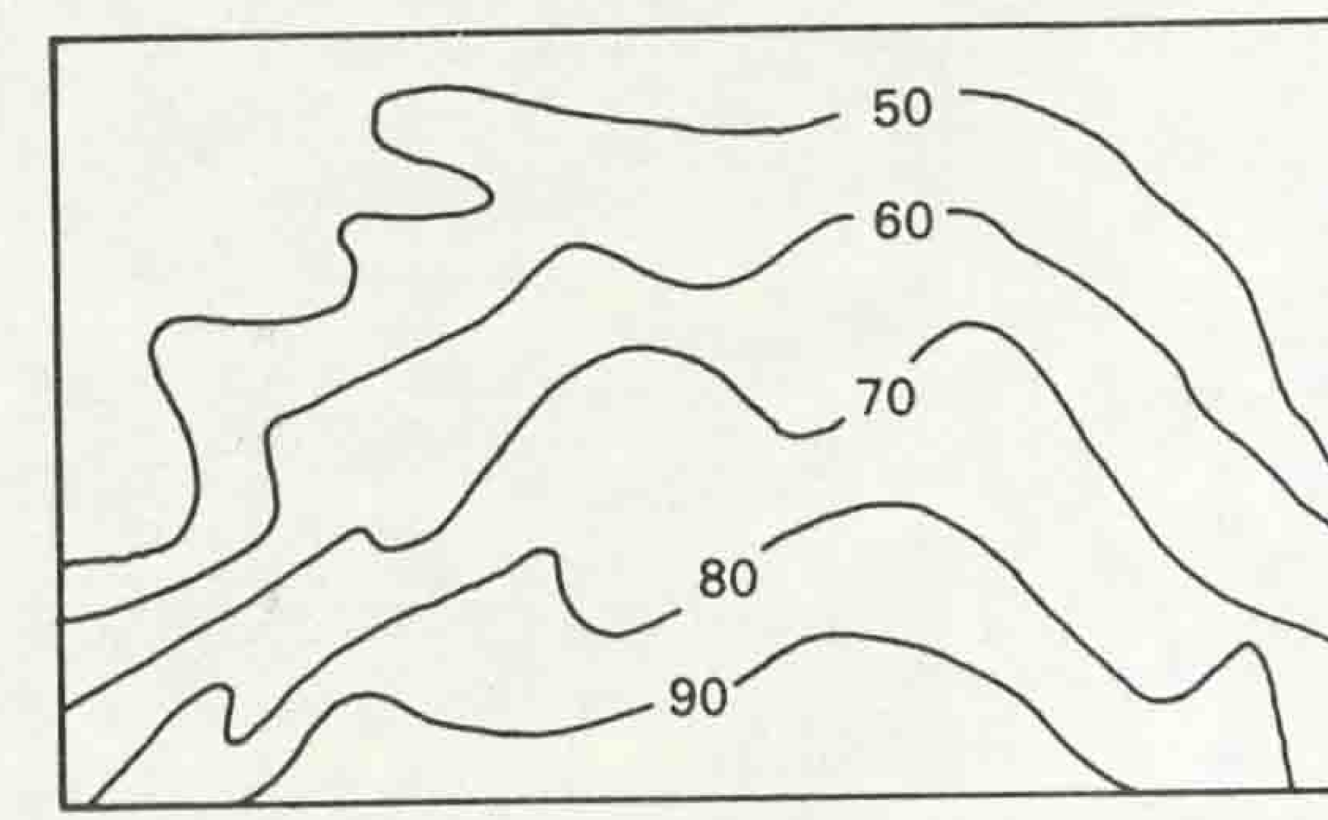


Figure 1.5 Typical relative percentage strength contours for a wall.

plan may, however, be expected to be random due to compaction and supply inconsistencies. Columns may be expected to be reasonably uniform except for a weaker zone in the top 300 mm or 20% of their depth (6).

1.4.2 *In-situ strength relative to standard specimens*

Likely strength variations within members have been described in section 1.4.1. If measured in-situ values are expressed as equivalent cube strengths, it will usually be found that they are less than the strengths of cubes made of concrete from the same mix which are compacted and cured in a 'standard' way. In-situ compaction and curing will vary widely, and other factors such as mixing, bleeding and susceptibility to impurities are difficult to predict. Nevertheless a general trend according to member type can be identified and the values given in Table 1.6 may be regarded as typical. The likely relationships between standard specimen strength and in-situ strength are also illustrated in Figure 1.6 for a typical structural concrete mix.

A "standard" cube is tested whilst saturated, and for ease of comparison the values of Table 1.6 are presented on this basis also. Dry cubes generally yield

Table 1.6 Comparison of in-situ and "standard" cube strengths

Member type	*Typical 28-day in-situ equivalent wet cube strength as % of "standard" cube strength*	
	Average	*Likely range*
Column	65%	55%–75%
Wall	65%	45%–95%
Beam	75%	60%–100%
Slab	50%	40%–60%

strengths which are approximately 10–15% higher, and this must be appreciated when interpreting in-situ strength test results. Cores will be tested whilst saturated under normal circumstances, and the above relationships will apply, but if the in-situ concrete is dry the figures for likely in-situ strength

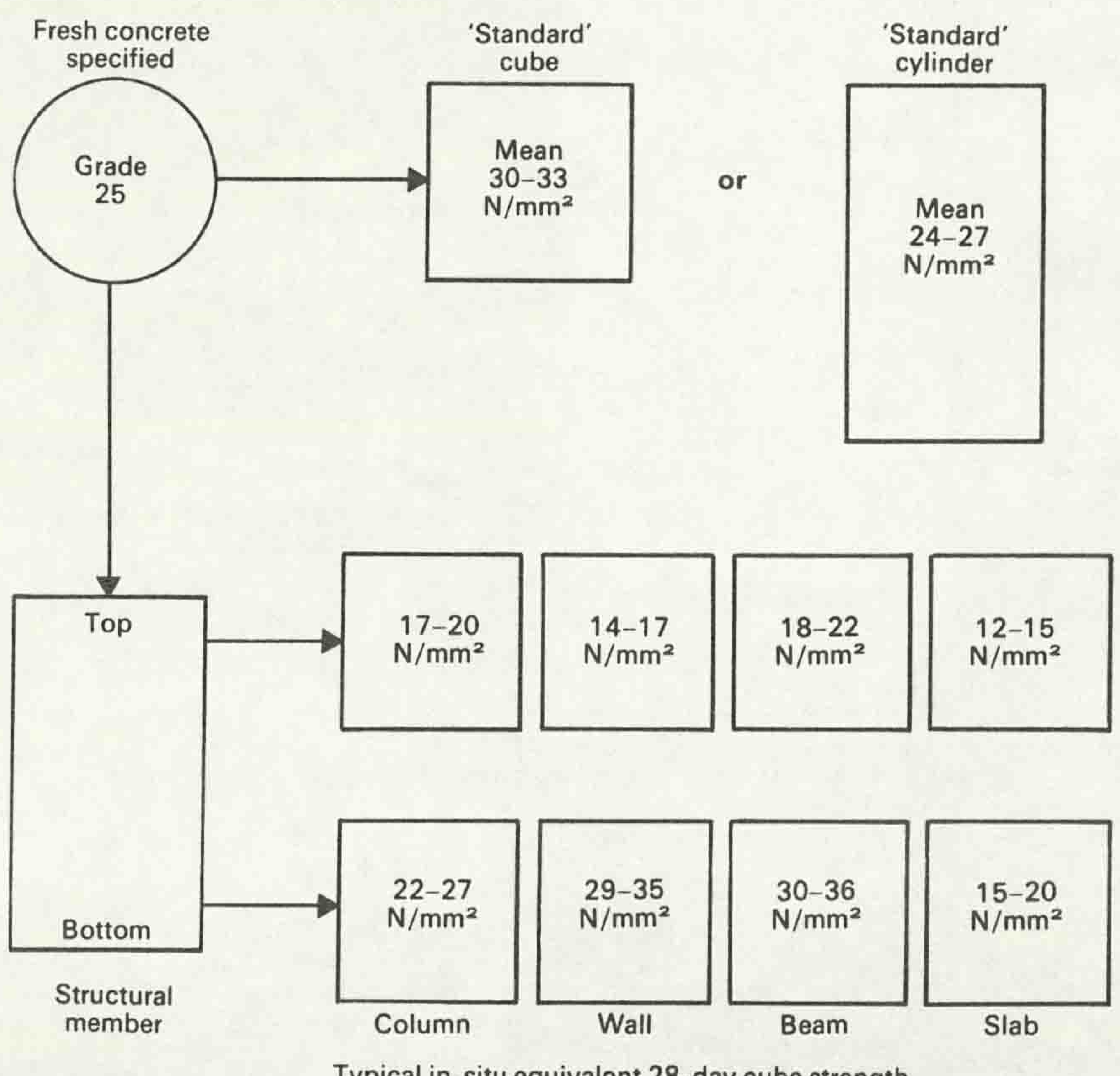


Figure 1.6 Typical relationship between standard specimen and in-situ strengths.

must be increased accordingly. Where non-destructive or semi-destructive methods are used in conjunction with a strength calibration, it is essential to know whether this calibration is based on wet or dry specimens. Another feature of such calibrations is the size of cube upon which they are based. Whilst design and specification are usually based on a 150 mm specimen, laboratory calibrations may sometimes be related to a 100 mm specimen which may be up to 4% stronger.

The age at which the concrete is tested is a further cause of differences between in-situ and "standard" values. Although "age correction" factors are given in Codes of Practice, care is needed when attempting to adjust in-situ measurements to an equivalent 28-day value. Developments in cement manufacture have tended towards yielding a high early strength with reduced long-term increases, and strength development also is largely dependent on curing. If concrete is naturally wet the strength may increase, but often concrete is dry in service and unlikely to make significant gains after 28 days.

1.5 Interpretation

Interpretation of in-situ test results may be considered in three distinct phases:

(1) Computation
(2) Examination of variability
(3) Calibration and/or application.

The emphasis will vary according to circumstances (detailed interpretative information is given in other chapters) but the principles will be similar whatever procedures are used, and these are outlined below. The examples of Appendix A further illustrate the application of those procedures to a number of commonly occurring situations.

1.5.1 *Computation of test results*

The amount of computation required to provide the appropriate parameter at a test location will vary according to the test method but will follow well-defined procedures. For example, cores must be corrected for length, orientation and reinforcement to yield an equivalent cube strength.

Pulse velocities must be calculated making due allowance for reinforcement, and pull-out, penetration resistance and surface hardness tests must be averaged to give a mean value. Attempts should not be made at this stage to invoke correlations with a property other than that measured directly. Chemical or similar tests will be evaluated to yield the appropriate parameter such as cement content or mix proportions. Load tests will usually be summarized in the form of load/deflection curves with moments evaluated for critical conditions, and creep and recovery indicated as described in Chapter 6.

1.5.2 *Examination of variability*

Whenever more than one test is carried out, a comparison of the variability of results can provide valuable information. Even where few results are available, (e.g. in load tests) these provide an indication of the uniformity of the construction and hence the significance of the results. In cases where more numerous results are available, as in non-destructive surveys, a study of variability can be used to define areas of differing quality. This can be coupled with a knowledge of test variability associated with the method to provide a measure of the construction standards and control used.

1.5.2.1 Graphical methods. "Contour" plots showing zones of equal strength (Figures 1.4 and 1.5) are valuable in locating areas of concrete which are

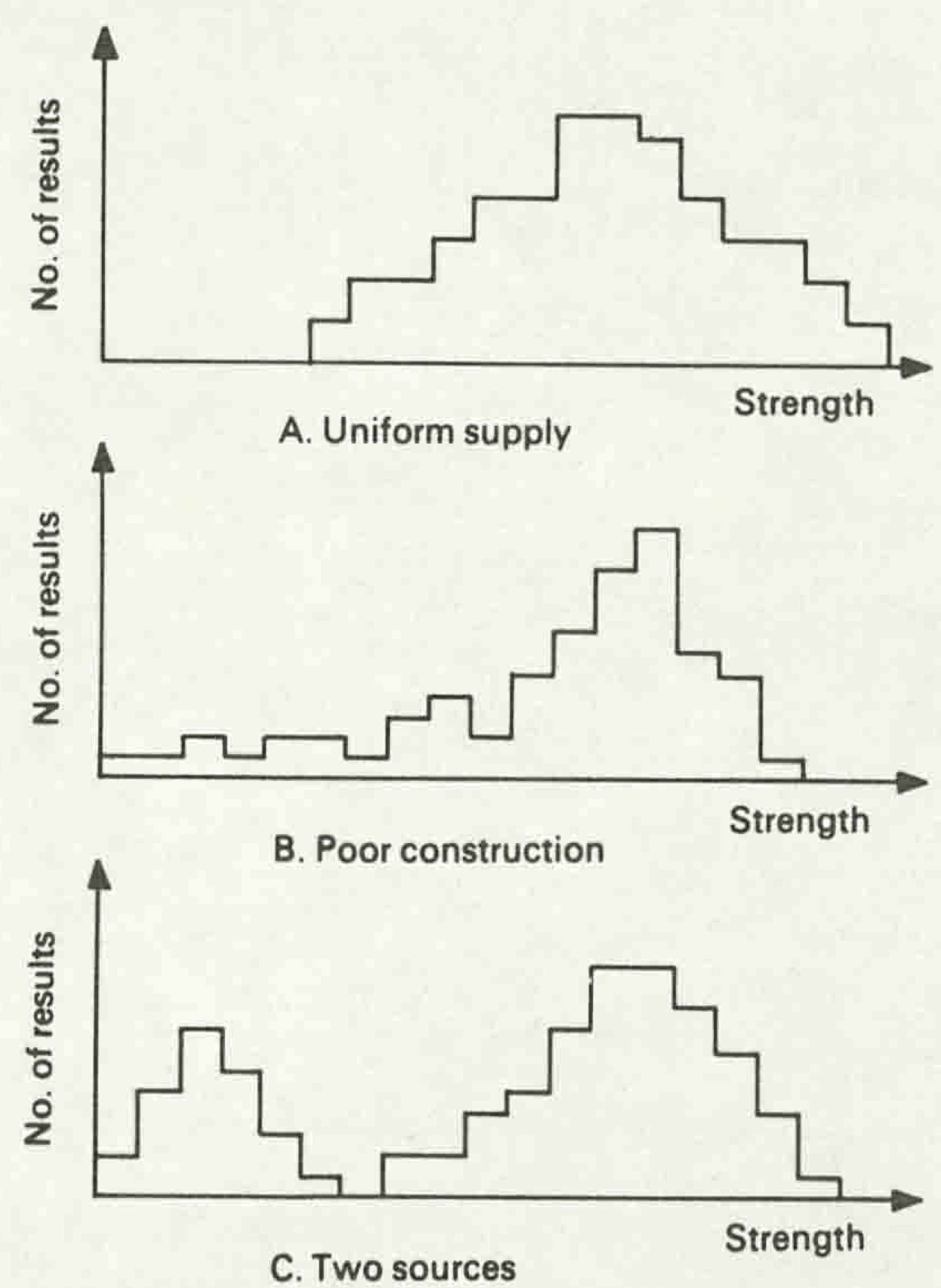


Figure 1.7 Typical histogram plots of in-situ test results.

abnormally high or low in strength relative to the remainder of the member. Such contours should be plotted directly on the basis of the parameter measured (e.g. pulse velocity) rather than after conversion to strength. Under normal circumstances the contour will follow well-defined patterns, and any departure from this pattern will indicate an area for concern. "Contour" plots are also valuable in showing the range of relative strengths within a member and may assist the location of further testing which may be of a more costly or damaging nature.

Concrete variability can also be usefully expressed as histograms, especially where a large number of results are available, as when large members are under test or where many similar members are being compared. Figure 1.7(*a*) shows a typical plot for well-constructed members using a uniform concrete supply. The parameter measured should be plotted directly, and although the spread will reflect member type and distribution of test locations as well as construction features, a single peak should emerge with an approximately normal distribution. A long "tail" as in Figure 1.7(*b*) suggests poor construction procedures, whilst twin peaks, 1.7(*c*), indicate two distinct qualities of concrete supply.

1.5.2.2 Numerical methods. Calculation of the coefficient of variation (equal to the standard deviation × 100/mean) of test results may provide valuable information about the construction standards employed. Table 1.7 contains typical values of coefficients of variation relating to the principal test methods which may be expected for a single site-made unit constructed from a number of batches. This information is based on the work of Tomsett (9), the author (5), (7), Concrete Society Report No. 11 (4) and other sources. Results for concrete from one batch would be expected to be correspondingly lower, whilst if a number of different member types are involved, the values may be expected to be higher. The values in Table 1.7 offer only a very approximate guide, but they should be sufficient to detect the presence of abnormal circumstances.

The coefficient of variation of concrete strength is not constant with varying strength for a given level of control because it is calculated using the average strength. Hence general relationships between coefficient of variation

Table 1.7 Typical coefficient of variation of tests results on an individual member corresponding to good quality construction

Core						
Large	*Small*	*Schmidt hammer*	*U.P.V.*	*Internal fracture*	*Pull-out*	*Windsor probe*
10%	15%	4%	2.5%	20%	7%	4%

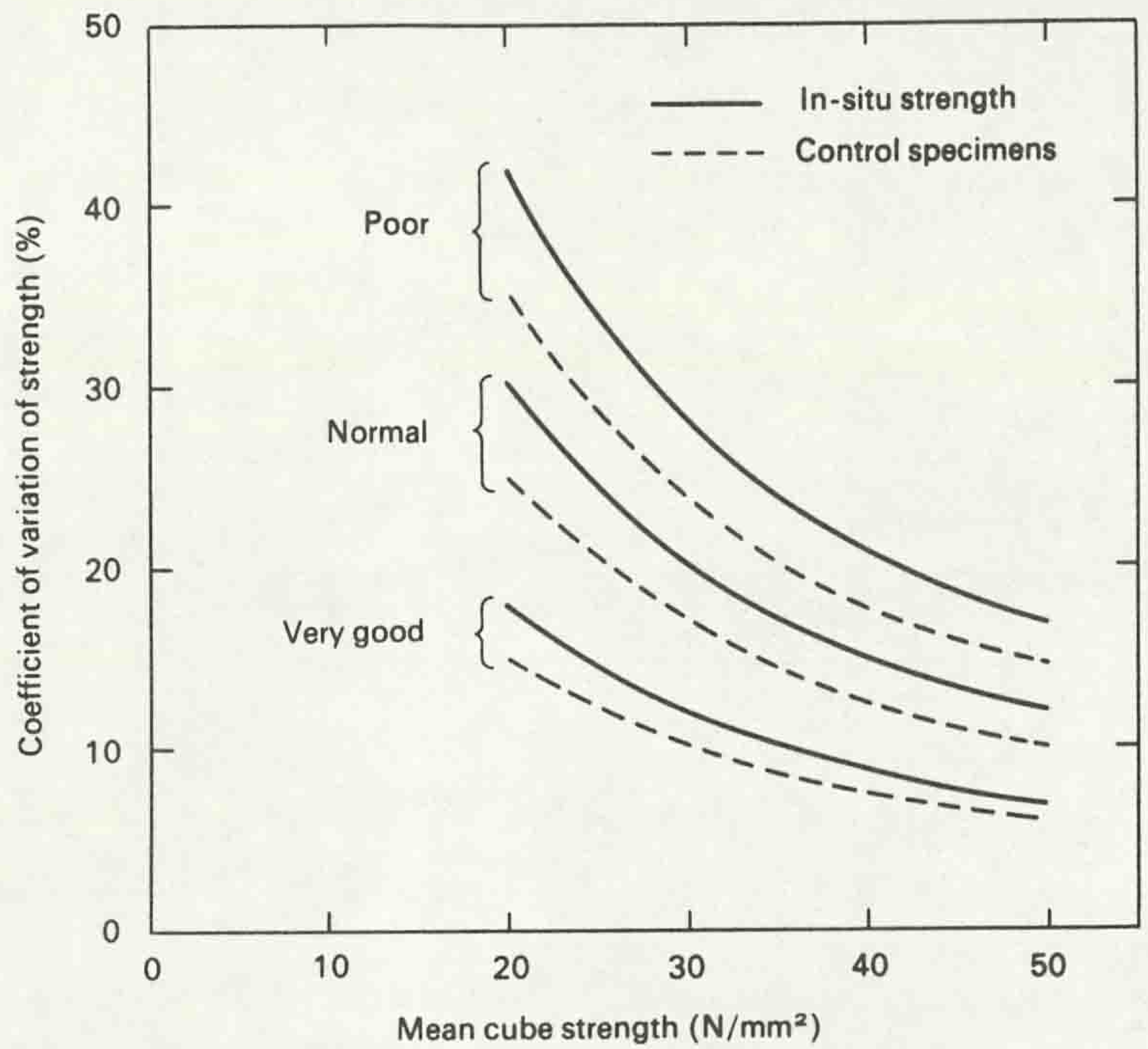


Figure 1.8 Coefficient of variation of test results related to concrete strength.

of measured concrete strength and level of construction quality should not be used. Figure 1.8 illustrates typical relationships for "standard" control cubes and in-situ strengths based on a variety of European and North American sources. From these values, anticipated standard deviations can be deduced (for example at 30 N/mm^2 mean in-situ strength, a standard deviation of $0.2 \times 30 = 6$ N/mm^2 is likely for normal quality construction) and hence confidence limits can be placed on the results obtained. Values such as those given later in Table 1.9 can be derived in this way, and strength accuracy predictions must make allowance for this as well as the accuracy of the test method.

1.5.3 *Calibration and application of tests*

The likely accuracies of calibration between measured test results and desired concrete properties are discussed in detail in the sections of this book dealing with each specific test. It is essential that the application of the results of in-situ testing takes account of such factors to determine their significance. Particular attention must be paid to the differences between laboratory

conditions (for which calibration curves will normally be produced) and site conditions. Concrete quality will vary throughout members and may not necessarily be identical in composition or condition to laboratory specimens. Also, the tests may not be so easy to perform or control due to adverse weather conditions, difficulties of access or lack of experience of operatives. Calibration of non-destructive and semi-destructive strength tests by means of cores from the in-situ concrete may often be possible and will reduce some of these differences.

Table 1.8 summarizes the maximum accuracies of in-situ strength prediction that can be hoped for under ideal conditions, with specific calibrations for the particular concrete mix in each case. If any factor varies from this ideal, the accuracies of prediction will be reduced, although at present there is little available information to permit this to be quantified. Wherever possible, test methods should be used which directly measure the required property, thereby reducing the uncertainties involved in calibration. Even in these situations, however, care must be taken to make a realistic assessment of the accuracy of the values emerging.

Table 1.8 Maximum accuracies of in-situ strength prediction

Method	*Max. likely strength accuracy (95% confidence limits)*
Cores (4)—"large"	± 6%
"small"	±18%
Ultrasonic pulse velocity	±20%
Windsor probe (3)	±20%
Schmidt hammer	±25%
Internal fracture (6)	±28%

1.5.3.1 Application to specifications. It is essential that the concrete tested is representative of the material under examination and this will influence the number and location of tests (section 1.3.3). Where some clearly defined property, such as cover or cement content, is being measured, it will generally be sufficient to compare measured results with the minimum specified value, bearing in mind the likely accuracy of the test. A small proportion of results marginally below the specified value may be acceptable, but the average for a number of locations should exceed the minimum limit. If the test has a low order of accuracy (for example cement content determination is unlikely to be better than ± 25 kg/m^3) the area of doubt concerning marginal results may be considerable. This is an unfortunate fact of life, although engineering judgements may perhaps be assisted by corroborative measurements of a different property.

Strength is the most common criterion for the judgement of compliance with specifications, and unfortunately the most difficult to resolve from in-situ testing because of the basic differences between in-situ concrete and the "standard" test specimens upon which most specifications are based (section 1.4.2). The number of in-situ test results will seldom be sufficient to permit a statistical assessment of the appropriate confidence limits (usually 95%), hence it is better to compare mean in-situ strength estimates with the expected mean "standard" test specimen result. This requires an estimate to be made of the likely standard deviation of standard specimens unless the value of target mean strength for the mix is known. The mean "standard" cube strength using British "limit state" design procedures is given by

$$f_{mean} = f_{cu} + 1.64s$$

where f_{cu} = characteristic strength of control cubes
s = standard deviation of control cubes.

The accuracy of this calculation will increase with the number of results available, 50 readings could be regarded as the minimum necessary to obtain a sufficiently accurate estimate of the actual standard deviation. If sufficient information is not available the values given in Table 1.9 may be used as a guide.

Table 1.9 Typical values of standard deviation of control cubes and in-situ concrete

Material control and construction	*Assumed std. devn. of control cube* (s) *N/mm²*	*Estimated std. devn. of in-situ concrete* (s′) *N/mm²*
Very good	3.0	3.5
Normal	5.0	6.0
Low	7.0	8.5

In theory it is possible to estimate the in-situ characteristic strength f'_{cu} from the measured in-situ values of the mean f'_{mean} and standard deviation s'. The values of s' given in Table 1.9 may be used in the absence of more specific data, but cannot be considered very reliable in view of within-member variations and the many variable constructional factors. If an in-situ characteristic strength is estimated it can be compared with the specified value, but this approach is not recommended unless numerous in-situ results are available.

Whichever approach is adopted, the comparison between in-situ and standard specimen strengths must allow for the type of differences indicated in Table 1.6 and Figure 1.6, and this is illustrated in the examples of Appendix A.

1.5.3.2 Application to design calculations. Measured in-situ values can be incorporated into calculations to assess structural adequacy. Whilst this may

occasionally relate to reinforcement quantities and location, or concrete properties such as permeability, in most instances it will be the concrete strength which is relevant. It is essential that the measured values relate to critical regions of the member under examination and tests must be planned with this in mind (section 1.3.3).

Calculations are generally based on minimum likely, or characteristic, "standard specimen" values modified by an appropriate factor of safety to give a minimum in-situ design value. In-situ measurements will yield directly an in-situ strength of the concrete tested and this must be related to a similar specimen type and size as the "standard" used in the calculations. If this concrete is from a critical location, it could be argued that the minimum measured value can be used directly as the design concrete strength with no further factors of safety applied. In practice, however, it is more appropriate to use the mean value from a number of test readings at critical locations, and to apply a factor of safety to this to account for test variability, possible lack of concrete homogeneity and future deterioration. The accuracy of strength prediction will vary according to the method used, but a factor of safety of 1.2 is recommended by BS 6089 (10) for general use. Providing the recommendations of section 1.3.3 have been followed when determining the number of readings, this value should be adequate. The application of this approach is illustrated in detail by the examples of Appendix A. If there is particular doubt about the reliability of the test results, or if the concrete tested is not from the critical location considered, then it may be necessary for the engineer to adopt a higher value for the factor of safety guided by the information contained in sections 1.4.1 and 1.4.2. Alternatively, other features discussed in section 1.4.2, including moisture condition and age, may possibly be used to justify a lower value for the factor of safety. The in-situ stress state and rate of loading may also be taken into account in critical circumstances.

1.6 Test combinations

All the test methods currently available for in-situ concrete assessment suffer limitations, and reliability, particularly from the point of view of strength assessment, is open to question. Considerably greater weight can be placed on results if corroborative values can be obtained from separate methods. Expense will usually restrict large-scale duplication to surface hardness and pulse velocity methods, but since these measure different properties of the concrete, confidence will be much increased if similar patterns of results emerge.

It has been suggested that the accuracy of absolute strength prediction can be improved by combined test results, and although data is limited, Facaoaru (11), Wiebenga (12), Macleod (13) and Samarin (14) have provided evidence to this effect for pulse velocity and rebound measurements. It would appear that this applies only when appropriate strength calibrations are available for both methods, but multiple regression equations can be developed involving pulse velocity and rebound number with compressive strength as the dependent variable. Correlation graphs may also be produced involving coefficients relating to various properties of the mix constituents as proposed by Facaoaru (11) on the basis of experience in Romania. The increased accuracy is attributed to the opposing influences of some of the many variables for each of the methods, and strength predictions to an accuracy of $\pm 15\%$ are claimed where direct calibration is available, or $\pm 20\%$ when the mix composition only is known.

Other combinations that have been proposed (15) include the use of dynamic modulus of elasticity, coupled with damping constant determined by laboratory resonance tests on prisms to predict strength, and ultrasonic pulse velocity combined with damping constant. These are of value only for laboratory specimens, although the combination of pulse velocity and pulse attenuation measurements can be used on site (16). The procedures are complex and require specialized equipment, and for practical purposes this approach must be considered as a research tool. The more common in-situ tests may certainly be combined in a variety of other ways but whilst valuable corroborative evidence may be gained it is unlikely that the accuracy of absolute strength predictions will be significantly improved.

Combinations of tests will also frequently be used for calibration purposes. This will usually involve core or destructive load tests in conjunction with non-destructive or semi-destructive methods which may then be more widely used.

Appendix A: Typical cases of test planning and interpretation of results

The engineer has complete and absolute authority as to whether concrete is condemned or accepted. The problem of testing, and interpretation of the results, will however be approached in a variety of ways—specifications, which will be used as the basis for decisions, vary widely and in some cases may legally empower the engineer to condemn concrete if the cubes fail, irrespective of the condition or quality of the in-situ concrete.

Many factors can however vitiate cube results including variations due to failure to observe the required standardized procedures for sampling, manufacture and curing of the cubes. Further errors may also be introduced by the testing operative or inaccuracies in the testing machine, although these should be checked by regular comparative reference testing. Whilst testing of the in-situ concrete eliminates most of these sources of error, specifications rarely mention in-situ strength and Codes of Practice do not define the in-situ strength required. BS CP110 (74), however, implies that an in place strength of $f_{cu}/1.5$ is expected for in-situ work by the adoption of a partial factor of safety of 1.5 when calculating the design concrete strength to use in calculations. If a design is based on some other Code of Practice this value may vary, but the basic principle that in-situ strength is recognised as being lower than standard cube specimen strength remains. It is to be hoped that engineers facing the problem of failed cubes will consider these aspects, as well as the in-situ requirements of the concrete and possible errors in the cube results, before taking decisions.

The following examples are included to assist the planning of tests and interpretation of results for this and other commonly occurring situations.

A1 28-day cubes fail (cube results suspect)

(a) *Problem*

28-day cubes from concrete used for in-situ beam construction are of low strength. Visual examination of the cubes suggests that they were poorly made.

(b) *Aims of testing*

The principal aim will be confirmation of specification compliance of the in-situ concrete, followed by determination of structural adequacy if non-compliance is indicated.

(c) *Proposals*

Schmidt hammer and/or ultrasonic pulse velocity testing of the beams is suggested, either by way of comparison with similar members of known acceptable standard cube strength, or to yield an absolute in-situ strength prediction based on a specifically prepared calibration for the particular mix. Visual comparison may also prove valuable. Tests should be located at mid-depth or spread evenly to provide a representative average value of in-situ strength for comparison with the specified value after application of safety factors. If doubt still exists then cores can be used, located to give representative values from the suspect concrete.

(d) *Interpretation*

If similar members are available for comparison with those that are suspect, the combined raw non-destructive test results should be plotted in histogram form and the overall coefficient of variation calculated to indicate the uniformity of concrete between members. The mean values for each group should also be calculated for comparison, bearing in mind possible minor variations due to age differences. If non-uniformity is indicated, relative strengths may be estimated from the mean non-destructive results for each group.

If similar satisfactory members are not available, or comparative non-destructive testing suggests borderline values, strength calibration charts for the particular mix may be used. If these are not available, or cannot be obtained, then cores cut to examine representative concrete of the suspect members should be used to estimate the equivalent "standard" cube strength (or potential cube strength).

(e) *Numerical example*

Specified 28-day characteristic cube strength = $30\,N/mm^2$. Schmidt hammer and UPV comparisons with similar beams cast one week earlier show only one peak in histogram form, and have the following values.

Schmidt hammer: mean rebound no. = 32
coefficient of variation = 6%
UPV mean velocity = 4.15 km/sec
coefficient of variation = 4%

Mean standard 28-day cube strengths for comparison beams = $34\,N/mm^2$,

hence

Estimated mean standard 28-day cube strength for suspect beams
= $32\,N/mm^2$ based on Schmidt hammer results
= $35\,N/mm^2$ based on ultrasonic pulse velocities

(using calibration curves of standard form.)

Whilst NDT results suggest only one supply and reasonable construction quality, the estimated mean equivalent standard 28-day cube strength for the suspect concrete is low in relation to the expected value of $30 + 1.64s$

where $s = 5\,N/mm^2$ (corresponding to "normal" standards—Table 1.9)
$= 38\,N/mm^2$.

The effects of age difference on these values can be assumed to lead to a small underestimate of true strength, but this cannot be relied upon. It appears therefore that whilst the mean value of strength for the suspect batches is above the minimum specified, the proportion of concrete likely to be below this value may be greater than normally permitted.

If further evidence is required cores should be used. These should be taken near mid-depth of a typical beam and at least four should be used to provide an estimate of potential strength as described in Appendix B. This estimate, which will have an accuracy of $\pm 15\%$, can then be compared with the absolute minimum specified ($0.85 \times 30 = 25.5\,N/mm^2$ for CP 110).

Core results:

estimated potential strength = 27
29
32
35

Mean = $31\,N/mm^2 \pm 4.5\,N/mm^2$.

Since the mean is above the characteristic specified value, and all results exceed the minimum acceptable value, it cannot be proved conclusively that the concrete does not meet the specification. Such results could well form part of an acceptable spread of values within a characteristic strength of $30\,N/mm^2$ and the concrete should not be rejected.

A2 28-day cubes fail (cube results genuine)

(a) *Problem*

A suspended floor slab has been cast from several batches and the 28-day cube strengths (6 cubes) are below the characteristic value required. There is no reason to suspect the validity of the cube results.

(b) *Aims of testing*

The initial aim will be to establish whether the low cubes are representative or relate to isolated substandard batches. The subsequent aim will then be to assess structural adequacy.

(c) *Proposals*

Visual inspection may indicate uniformity or otherwise of the slab in the first instance. This can be followed by Schmidt hammer tests on the soffit to confirm uniformity. Direct ultrasonic tests may prove difficult, and with indirect readings on the soffit being unreliable cores should be taken from typical zones. In cases of doubt, in-situ load testing may be necessary.

(d) *Interpretation*

Schmidt hammer readings should be taken on a regular grid, and plotted on a "contour" plan to indicate the degree of uniformity. A histogram plot may also be worthwhile to provide confirmation of this. The results from cores can be used to estimate an average actual in-situ strength which can then be related to the required design strength with an allowance for the likely standard deviation of in-situ results.

(e) *Numerical example*

Specified characteristic 28-day cube strength (f_{cu}) = $40\,N/mm^2$
Mean "standard" 28-day cube strength = $38\,N/mm^2 \pm 2\,N/mm^2$
Mix design mean strength = $46\,N/mm^2$.

Visual inspection and non-destructive tests results suggest uniform construction across the slab.

Average estimated in-situ "actual" cube strength from six cores

$$= 30 \pm 1.5\,\text{N/mm}^2.$$

$$\text{Estimated characteristic in-situ strength range} = \begin{cases} 31.5 - 1.64s'\,\text{N/mm}^2 \\ 28.5 - 1.64s'\,\text{N/mm}^2. \end{cases}$$

Adopt estimated in-situ standard deviation (s') = 4.5 N/mm^2

based on scatter of Schmidt hammer results and past site cube records which shows "good" control (see Table 1.9).

Hence,

$$\text{estimated characteristic in-situ strength} = \begin{cases} 31.5 - 7.5 = 24\,\text{N/mm}^2 \\ 28.5 - 7.5 = 21\,\text{N/mm}^2 \end{cases}$$

If the design is to CP 110 (74), the partial factor of safety on concrete strength is 1.5, hence minimum acceptable design strength = 40/1.5 = 26N/mm^2. Since the estimated range of characteristic in-situ strength lies below the minimum design strength, the concrete must be considered unacceptable. If the slab is to be permanently dry, the estimated in-situ values may be increased by 10%, but this factor will not be sufficient to accept the concrete unless the slab is not critically stressed. It is recommended that an in-situ load test be undertaken to establish directly the serviceability behaviour of the slab.

A3 Cubes non-existent for new structure

(a) *Problem*

A large number of columns have been cast, and the cubes lost.

(b) *Aims of testing*

The principal aim will generally be confirmation of acceptability of the in-situ concrete from the point of view of strength and durability.

(c) *Proposals*

A comprehensive comparative survey, using surface hardness and/or ultrasonic pulse velocity. Visual inspection may also indicate lack of uniformity. Plot raw results to indicate patterns and then follow up with a limited number of cores at points of apparently lowest and highest strength, unless reliable calibrations for the mix are available or can be obtained. If similar members

are available of concrete which is known to be acceptable it may be adequate to rely on non-destructive comparisons with these, using cores only in cases of extreme doubt. Reserve crushed cores for chemical analysis to determine the cement content if strengths indicate durability doubts.

(d) *Interpretation*

It is important that the non-destructive results to be used comparatively are taken at comparable points in relation to the members tested. This is because of the likely within-member strength variations, and measurements should preferably be taken at points of expected lowest strength (i.e. near the top of the columns). Sufficient readings should be taken to encompass the various batches of concrete that may have been used.

Results should be plotted in histogram form to detect the weakest areas; coefficients of variation may also provide valuable confirmation of construction uniformity. The cores should provide values for minimum strength to be compared with a calculated minimum acceptable value from the design, as well as a rough calibration for the non-destructive tests. If attempts are to be made to relate results to specifications, the likely within member variations and in-situ/standard specimen strength must not be overlooked. Under normal circumstances a minimum in-situ strength of characteristic/1.5 would be acceptable for design to BS CP110 (74), whilst an even lower value may be adequate for low stress areas, subject to adequate durability as indicated by cement content.

(e) *Numerical example*

Specified characteristic cube strength (f_{cu}) = 30 N/mm^2.

For design to BS CP110 (74) assume

$$\text{minimum acceptable in-situ strength} = \frac{f_{cu}}{1.5} = 20\,\text{N/mm}^2.$$

For four cores taken to correspond with lowest pulse velocities and rebound numbers:

estimated actual cube strengths	20.5 N/mm^2
	25.0 N/mm^2
	22.5 N/mm^2
	21.0 N/mm^2
Mean	22.0 N/mm^2

$\therefore$ Estimated minimum in-situ strength = 22 N/mm^2 ± 1.5 N/mm^2.

Hence the mean and all results are above minimum acceptable value, and the concrete will be considered adequate. It will follow that the remainder of the concrete is also acceptable since these results relate to the worst locations.

Note (1): If either individual results or the mean estimated in-situ strength are below the minimum acceptable as calculated above (based on BS CP110) detailed consideration should be given to design stress levels, and service moisture conditions.

Note (2): If the concrete strength is critical to the design, or if calibrations have been used with surface hardness or UPV results to estimate strength without cores, it may be appropriate to include a factor of safety to account for this, e.g. maximum acceptable design stress = 22/1.2 = 18 N/mm^2 based on the factor of safety of 1.2 recommended by BS 6089 (10).

A4 Cubes non-existent for existing structure

(a) *Aims of testing*

The aim of testing will be to provide a concrete strength estimate for use in design calculations relating to a proposed modification of the structure.

(b) *Proposals*

Survey by ultrasonic pulse velocity or Windsor probe, correlated with a limited number of cores according to practical limitations. If cores are impossible, survey by internal fracture test. Tests to be spread as representatively as possible over members under examination.

(c) *Interpretation*

Use mean estimated in-situ cube strength to obtain a value of design strength which takes account of the standard of construction quality as well as uncertainties about the adequacy of the in-situ test data.

(d) *Numerical example*

Estimated mean in-situ cube strength = 25 N/mm^2.
Assumed mean "standard" cube strength = $25 \times 1.5 = 37.5$ N/mm^2

for "normal" construction quality (unless evidence from scatter of test results suggests otherwise), standard deviation of control cubes estimated at 5 N/mm^2 (Table 1.9).

Hence estimated characteristic "standard" cube strength $= 37.5 - 1.64 \times 5$
$= 29$ N/mm^2.

Allowance should be made for errors in test data by a factor of safety (1.2 suggested by BS 6089 (10)).

Hence maximum design stress using a partial factor of safety of 1.5 on concrete strength for design to CP 110 (74) equals:

$$\frac{29}{1.2 \times 1.5} = 16\ \mathrm{N/mm^2}.$$

Alternatively, if the test results are taken to correspond to locations of lowest anticipated strength:

Maximum design stress = estimated minimum in-situ cube strength, given by mean of test results/1.2.

A5 Surface cracking

(a) *Problem*

The wing wall to a highway bridge abutment shows random surface cracks and spalling several years after construction.

(b) *Aims of testing*

The principal aim will be identification of the course of deterioration followed by an assessment of present and future serviceability. Apportionment of blame may follow.

(c) *Proposals*

Visual inspection of crack patterns, and their development with time, may permit preliminary classification of cause as (*a*) structural actions, (*b*) shrinkage, or (*c*) material deterioration. This may be followed by strength assessment as in A2 if structural actions are suspected, or chemical/petrographic testing if material deterioration is likely. Cores may conveniently be used to provide suitable samples and should be taken from the areas most seriously affected. Chemical testing to detect chlorides or sulphates will be selected according to the crack pattern, whilst microscopic examination can check for frost action, alkali/aggregate reaction and entrained air content.

Serviceability will be determined on the basis of the extent of deterioration and the ability to prevent worsening of the situation.

(d) *Interpretation*

Reference to Table 1.6 will assist preliminary identification. Shrinkage cracks are likely to occur at an early age and follow a recognizable pattern, as do cracks due to structural actions. Material deterioration is therefore indicated in this case, and may be due to chemical attack from internal or external sources or due to frost action. Chloride attack is unlikely since the cracks do not follow the pattern of reinforcement, thus initially test for sulphate and cement content. If the results of these tests indicate acceptable levels, petrographic examination will be necessary to attempt to identify aggregate/alkali attack or frost action. If frost action is indicated, micrometric examination will yield an estimate of the entrained air content for comparison with the specified value.

If future deterioration can be prevented by protection of the concrete from the source of attack, this should be implemented after such cutting out and making good as may be necessary. If the source of deterioration is internal and not of a localized nature it may prove necessary to replace the member once it reaches a condition of being unfit for use.

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Paper 17

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'Features of Assessment of

Pre-cast Pre-tensioned Beams in Structures'

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'FEATURES OF ASSESSMENT OF PRECAST PRETENSIONED BEAMS IN STRUCTURES'

by

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SYNOPSIS

Problems of in-situ assessment of small section pretensioned beams are examined for situations in which concrete strength deterioration is suspected. A planned approach involving non-destructive, semi-destructive and destructive methods is proposed and illustrated by case histories of High Alumina Cement investigations. Examples of testing to determine composite actions are described and the importance of 'non-structural' finishes also illustrated. Although existing test methods do not yield precise concrete strengths it is shown that they permit reasonable estimates of existing member strengths to be made. The many design and constructional inadequacies encountered in these and similar investigations emphasise the need for routine inspection of concrete structures, so that combinations of faults may be detected before failures occur.

INTRODUCTION

During the late 1950's and 1960's High Alumina Cement (H.A.C) was used extensively in the U.K. by precasting manufacturers in the construction of long-line pretensioned units. Despite warnings (1) it was not until 1974 that the severity of the problem of chemical decomposition of the unstable hydration products from this type of cement was fully appreciated. It was then considered necessary to examine the majority of structures incorporating the material. The Author was involved both in the development and application of non-destructive testing methods to deal with this problem, and also in very many in-situ investigations of structures incorporating beams of this type. These investigations highlighted many general problems of assessment, especially of small section pretensioned beams, under service conditions.

It became apparent that a logical planned approach to any in-situ assessment is vital, and the procedure illustrated in Figure 1 is proposed to minimise effort and costs. In pursuing such a program it is essential that the first step is to establish exactly what information is required from the assessment, i.e. Materials properties, Member strength or Overall structure strength, and whether these are short or long-term considerations. Subsequent operations will normally be followed in sequence, as necessary with the purposes indicated.

PROBLEMS OF HIGH ALUMINA CEMENT CONCRETE

H.A.C concrete was popular with pre-casters because of the very high 24 hour strength that can be achieved, and although it was known that a long-term strength reduction due to decomposition or 'conversion' to stable products took place, little was known of the details or extent of such changes. It is now established (2) that the chemical changes are temperature and moisture dependent and that the resulting residual strength may be as low as 25% of the 24 hour value under warm damp conditions. The initial water/cement ratio is also a contributory factor (3) and the problem is further aggravated by the very low resistance to chemical attack of the 'converted' concrete.

Visual inspection was found to be of little value in assessing the state of the concrete since although a colour change was sometimes apparent this could not be relied upon. It has been shown however (2) that the extent of the conversion in a member can be assessed by Differential Thermal Analysis (D.T.A), but this in itself gives no indication of concrete strength. Residual strength estimation must account for both the extent and rate of conversion, and is virtually impossible to assess from D.T.A results in most situations. It was necessary therefore to make use of tests which give a more direct measurement of concrete strength.

H.A.C was typically used for small section 'standard' beams (Figure 2) in which the number of prestressing wires could vary according to the design requirements. These were either used as isolated members, supporting light-weight roofing, or more commonly in conjunction with lightweight blocks to form floors or roofs. Many problems of access, visual inspection and identification were thus encountered in addition to the difficulties of application of non-destructive tests to members of this type in service.

APPLICATION OF TEST PROCEDURES TO PRETENSIONED BEAMS

Non-Destructive

Chemical testing, whilst yielding many details of a mix does not permit an accurate estimation of water/cement ratio and is hence of little value in strength prediction. The major use of chemical testing is thus in assessment of chemical attack, and the indication of areas where strength reduction is likely, rather than in the direct assessment of concrete strength.

Surface hardness tests are of very limited value for testing concrete which is more than a few months old because of surface carbonation effects, and this applies to both Portland Cement and H.A.C Concretes. Ultrasonic Pulse Velocity testing potentially offers the most efficient method of examination of the interior of a concrete member. The approach is well established for reinforced concrete members, but application to small section prestressed members introduces additional problems. Although it has been found that the presence of small diameter prestressing wires have a negligible effect on pulse velocity the stress history will influence results when stresses exceed 1/3 cube strength (4), and this must be taken into account when interpreting results.

The influence of member size on measured pulse velocities must be considered together with the difficulty of access to obtain a direct reading in many situations. Laboratory tests show that for portable testing equipment currently in use in the U.K., using 52 k Hz. transducers, a minimum path length of 100mm. is essential to obtain reliable results (4). The path width should also ideally be greater than 50mm. It will be seen (Figure 2) that the only way that the dimensional requirements can be met is to take readings horizontally across flanges or vertically through the web. Screeds and other finishes usually rule out the latter, thus readings must normally be taken across flanges and laboratory tests (4) confirm that transducer overhang and path width effects are small for sections of this type.

In a situation where concrete strength deterioration is suspected, it is obviously preferable to measure the compressive flange values. However, when the beams are used in conjunction with infill blocks, it is usually only possible to break out sufficient blocks to take readings across bottom flanges. The use of indirect readings along exposed soffits is of little value when attempting strength assessment if flexural cracking has occurred, although the author is currently examining the use of this approach to comparative crack depth assessment, and subsequent vulnerability to chemical attack.

Provided details of the mix are known, calibration of ultrasonics for particular aggregate characteristics can be easily obtained in the laboratory with standard control specimens for Portland Cement concretes. In the case of H.A.C, difficulties were experienced in reproducing the physical changes due to conversion artificially in the laboratory, hence calibration charts could not be obtained in this way. It was necessary therefore, to use cores from beams which had converted 'naturally' to develop calibration charts. Where possible ultrasonic readings were taken across member flanges at core positions prior to cutting, although in some cases it was only possible to take readings on cores. The scatter inherent in testing small cores leads to a wide band of results, with 95% confidence limits on predicted strength of the order of ± 50% (5).

Semi-Destructive

Concrete strength estimation using cores in excess of 100mm. diameter may be expected to yield reasonably reliable results. However, it is seldom practicable to obtain cores of this size with a satisfactory length/diameter ratio from pretensioned beams in service. In such situations where cores are considered essential recourse must be made to smaller diameter specimens.

The reliability of these has been shown to be considerably worse than for the larger specimens (6) and to be dependent on the maximum aggregate size. Nevertheless it is demonstrated in Case History A that results of cut cores from small section members can be valuable.

The concept of concrete strength estimation by measuring the force required to pull out a fixing is not new, but recent attention has been concentrated on a version specifically intended for insitu usage on small section members. A 6 mm. expanding sleeve wedge anchor bolt is inserted into a hole of similar size drilled with a masonry drill in the exposed surface of the member. The sleeve is set at a standard depth of 20mm, and the force necessary to pull the bolt from the concrete is measured. Although the Building Research Establishment (7) recommend the use of a torquemeter, the Author has developed an alternative approach providing a direct pull to the bolt through a proving ring. The potential advantages of this test are that only one flat surface of the member need be exposed, and preliminary evaluation indicates that strength calibration is less sensitive to aggregate properties and proportions than Ultrasonics. However Figure 3 shows laboratory results for mixes with 10mm. gravel aggregates, indicating a greater scatter for a particular aggregate type and size than expected for Ultrasonics. These results give 95% confidence limits on predicted strength of ± 23% compared with ± 16% obtained from ultrasonic tests on the same mixes. This is obviously a surface zone test, and the significance of this together with the influence of concrete stress history are still under investigation.

Load Testing

Load testing of members is expensive and inconvenient. However, insitu overload tests may provide the only practical method of confirming the ability of members to carry their intended loading. Where precast beams form part of a composite member, the anticipated behaviour may be calculated reasonably accurately, but where 'non-structural' finishes are present estimation of their partial composite action and load transfer capacity poses many problems. Where there is considerable lateral flexibility it is possible to concentrate loads on one member as in Case History B. In the case of floors, which are generally much stiffer, it will be necessary to have either very large loaded areas, or isolation of an individual element of the structure by saw cutting, to ensure that the members under test are fully-loaded. This latter approach is obviously likely to weaken the structure and is unlikely to be used except in cases of extreme doubt, or where large numbers of similar members are involved, as in Case History C.

Whilst this form of test may instill confidence in the minds of building users, the value in assessing long-term adequacy is difficult to justify when dealing with a time dependent deterioration. Collapse load tests, preferably in a laboratory, yield considerably more information and can provide a basis for calibration of non-destructive tests which may then be used for on site comparison of similar members. In the early stages of H.A.C investigations many such tests were performed on suspect members removed from structures, and the resulting data later provided a basis for assessment of similar members both practically and theoretically.

TESTING CASE HISTORIES

Case History A

A major investigation in which the full range of test procedures was necessary involved several thousand identical 200mm. deep T-section purlins. Each spanned 5.8m. between pre-cast reinforced concrete portal frames and supported asbestos-cement corrugated panels to form the roof to a large fertilizer store. A small group of beams collapsed overnight, without warning, apparently by flexural shear failure. It was noted that the concrete in these beams, and their neighbours, was very crumbly with extensive signs of chemical attack. It was further observed that these beams were located close to heavy transport access doors which were normally open in the day-time and that rain was often blown in. Chemical analysis on the failed beams indicated sulphate attack, and although visual inspection indicated no signs of serviceability distress elsewhere in the building, it was noted that most beams were coated with fertilizer dust. It was concluded that moisture blown in at the doors had assisted attack of the concrete by soluble chemicals in the fertilizer dust. Ultrasonic tests were taken on a very small random sample of visually sound beams and direct readings across the top flanges yielded consistent results, averaging 4.40 km/sec. H.A.C calibration charts prepared by the Author indicated a likely average compressive strength of 40 N/mm^2, and calculations of Ultimate Moment of Resistance and Shear Resistance indicated satisfactory margins of safety for the design loadings on this basis.

In view of the large number of beams involved, all areas were subjected to a detailed visual inspection, and a selection of beams removed and tested to destruction in the laboratory to verify the predictions. These generally exhibited satisfactory elastic behaviour up to the design working load, but subsequently suffered collapse at a range of loads. Small 44mm. cores were cut from the intact regions and tested in compression, and Figure 4 shows the relationship between measured Collapse Moment and average actual compressive cube strengths based on these cores. The theoretical curve is based on a rectangular parabolic stress block, as proposed by British Standard Code of Practice, CP.110 (8), with materials factors of safety removed and using an estimated average value of steel yield stress. One reason for the theory providing a lower bound is that plotted concrete strengths are estimates of the actual strength of the concrete rather than the 'potential' strength of standard control specimens, which will invariably be higher and upon which design theory is based. These results demonstrate that despite the problems of interpretation of small core results, they can provide a useful guide to the engineer.

Correlation between visual assessment of surface deterioration and measured collapse strength was good, and low ultrasonic readings were obtained for the most seriously affected beams. D.T.A. tests indicated that conversion was at a moderately advanced stage, and it was concluded that the majority of beams, although fairly highly converted, were quite adequate for their intended purpose. Those that were not, could be detected by careful visual inspection of surface conditions backed up by comparison of measured pulse velocities with a lower safe limit which was found to be 3.85 km/sec. Provided beams remained dry, no general deterioration was likely, but those near to entrances were particularly vulnerable to future deterioration and were replaced. Regular checking and roof maintenance, to prevent leaks, thus became essential to ensure the safety of those remaining. These findings were confirmed by similar laboratory testing of a further small random sample of beams approximately two years later, in which it was found that 'conversion' had continued, but that strength results were only marginally reduced, and remained well within acceptable limits.

Case History B

710mm. deep pre-cast pretensioned I-section roof beams at 9.1m. centres spanned 15.5m. supporting precast lightweight concrete roof panels with waterproofing screed and finishes.

D.T.A. indicated that these large H.A.C beams were highly converted and ultrasonic readings across the flanges of the beams showed likely average compressive cube strengths of between 30 and 40 N/mm². A few small diameter cores drilled vertically into the centre of the top flanges confirmed this with an average of 32 N/mm². An estimate of overall load factor based on this indicated a value of 1.5 on working loads if no composite action was assumed. This was considered to be inadequate since these beams spanned over lecture theatres, in which large numbers of people were likely to be assembled. An in-situ load test was thus proposed to confirm acceptable working load behaviour and to assess the extent of composite action that could be assumed.

Steel weights were used to provide a concentrated load directly above one beam throughout its length. Deflection readings wre taken for this and the adjacent beams and from comparison of these deflections it was estimated that 26% of the load was transferred to adjacent beams. This left the beam under test supporting only 74% of the test load, which had been calculated as 1.25 x live load + 0.25 x dead load. The estimated test load carried by the beam was equivalent to 1.7 x design live load, and was considered to be adequate in view of the good recovery that was achieved after the load had been sustained for 24 hours.

The mid-span deflection measured was only 25% of that which would have been expected on this basis for the isolated beam, thus it was concluded that considerable composite action was occurring and that the true ultimate strength of the roof was considerably in excess of that for isolated beams. In view of this, coupled with the knowledge that conversion was virtually complete, the structure was cleared subject to regular long-term inspection and monitoring of levels.

Case History C

A major development involved large areas of floor construction, similar to that in Figure 2, spanning 5.5m. The screed was of structural concrete and the precast beams contained mild steel shear connectors projecting from their upper surface. Laboratory load tests on similar individual beams confirmed variable strengths which had first been indicated by in-situ ultrasonic results. It could be shown, however, that the reduction in concrete strength would have little overall effect provided that the intended composite action could be relied upon, and an in-situ load test was performed to confirm this. An area of floor 750mm. wide was isolated by saw cuts parallel to the beams and passing through slab and infill blocks over the entire span. A test load of 2 x design imposed load was provided by bricks uniformly distributed along this strip, but concentrated above the two beams included in the test section. Measured deflections were found to be about 60% of calculated values assuming full composite action between beams and slab, and this discrepancy was attributed partially to some end fixity of units and also to the stiffening effect of the hollow infill blocks.

ASSESSMENT BY CALCULATION

Where it is required to assess structures by calculation alone, it has been demonstrated that calculation on the basis of estimated actual concrete strength yields a reasonable lower bound solution for isolated beams. This has been confirmed by many other test results for both X-section and rectangular beams.

Allowances for partial composite action are difficult to determine, expecially in the case of hollow block construction (Figure 2), and will depend largely on whether the unit is under or over-reinforced at failure. Full-scale tests to destruction on slabs of this type have been reported (9) and a maximum strength enhancement factor of 1.25 is proposed (10) by the Building Regulations Advisory Committee appointed to examine the H.A.C problem.

In most H.A.C situations, calculations of long-term adequacy were based on a likely minimum concrete strength proposed by the above Committee, in view of the particular time related phenomenon. D.T.A. and Ultrasonic Tests enabled an assessment of each individual case relative to this and hence modification to standard calculations if necessary. Such calculations enabled the majority of cases to be cleared as satisfactory, but the number of cases relying on 'non-structural' finishes enhancement factors was high. In some instances, as in Case History D, the absence of such finishes led to expensive remedial measures.

Case History D

Office floor construction was similar to Figure 2 but without the screed. Floor finishes consisted of polythene sheeting, sand, hardboard and flexible floor tiles, and thus contributed no strength. Visual and non-destructive testing indicated that whilst some deterioration had occurred it was not severe, and it was thus necessary to assess long-term strength using the Building Regulations Advisory Committee recommendations. These indicated that long-term member strength was inadequate if there was no composite action from finishes, and remedial measures were thus necessary. The solution adopted was a 50mm. reinforced concrete slab cast over the entire floor after removal of existing finishes, bond between this and the existing rough concrete and block surface enabling adequate composite action to occur.

GENERAL OBSERVATIONS AND DISCUSSION

Whilst investigating many structures it was apparent that service deflection, or lack of it, was often unrelated to concrete strength. Whilst weakened rectangular beams generally showed considerable deflections, this was not the case for lighter X or I-section members, especially when finishes were present. The importance of non-structural finishes has been highlighted in Case Histories B and D, and it is thought likely that the hard surface skin also has a considerable stiffening effect on members of this type, even though strength and elastic modulus of the interior may have decreased. Some cases were found where beams were showing considerable deflection despite adequate strength, but these were shown to be coupled with excessive design span/depth ratios.

If time and costs of in-situ investigation are to be kept down it is essential that a planned testing approach is adopted, partially due to the difficulties of assessing concrete strength from the non-destructive tests currently available. It must be remembered that testing yields only information about the short-term condition of a structure. Long-term prospects can only be predicted by the Engineer applying his experience and judgement to results.

Problems of unexpected chemical attack have been demonstrated in Case History A. Other cases range from beams in chemical testing laboratories which were visibly disintegrating under the attack of air borne fumes, to the common case of roof leakage enabling chemicals leached from Portland Cement screeds to attack converted H.A.C. beams. Other unexpected service conditions include warm damp conditions in roofs of swimming pools, Laundries, chip shops and kitchens. This was often aggravated by solar heating on flat dark surfaces, and by suspended ceilings creating voids in which high ambient temperatures were observed.

Construction faults such as inadequate seatings for precast units, or unauthorised holes for services were also frequently encountered. Structures seldom fail in service from one cause alone but when strength loss is coupled with inadequate design standards, poor workmanship or unforeseen service conditions then trouble may occur. The frequency with which general deficiencies were found emphasises the importance of regular routine inspections of precast pretensioned units, irrespective of cement type, as part of any maintenance programme.

CONCLUSIONS

(1) In-service deflection is an unreliable guide to concrete strength deterioration.

(2) Ultrasonic testing can be used on small section prestressed concrete beams for comparison and strength estimation provided that adequate calibration can be obtained.

(3) Concrete strength estimates from small cores can be used successfully to provide a conservative estimate of member strength.

(4) Pull-out testing may potentially offer practical advantages and require less specific calibration than other non-destructive tests.

(5) In-situ load tests are useful in establishing the extent of both designed and unintended composite actions.

(6) Destructive testing in the laboratory is worthwhile for calibration purposes if large numbers of similar members are involved.

(7) Routine inspection of precast pretensioned beams should be included in maintenance programmes.

ACKNOWLEDGEMENT

The investigations described in Case Histories B, C and D were undertaken in collaboration with Professor F. Sawko of the Civil Engineering Department at Liverpool University.

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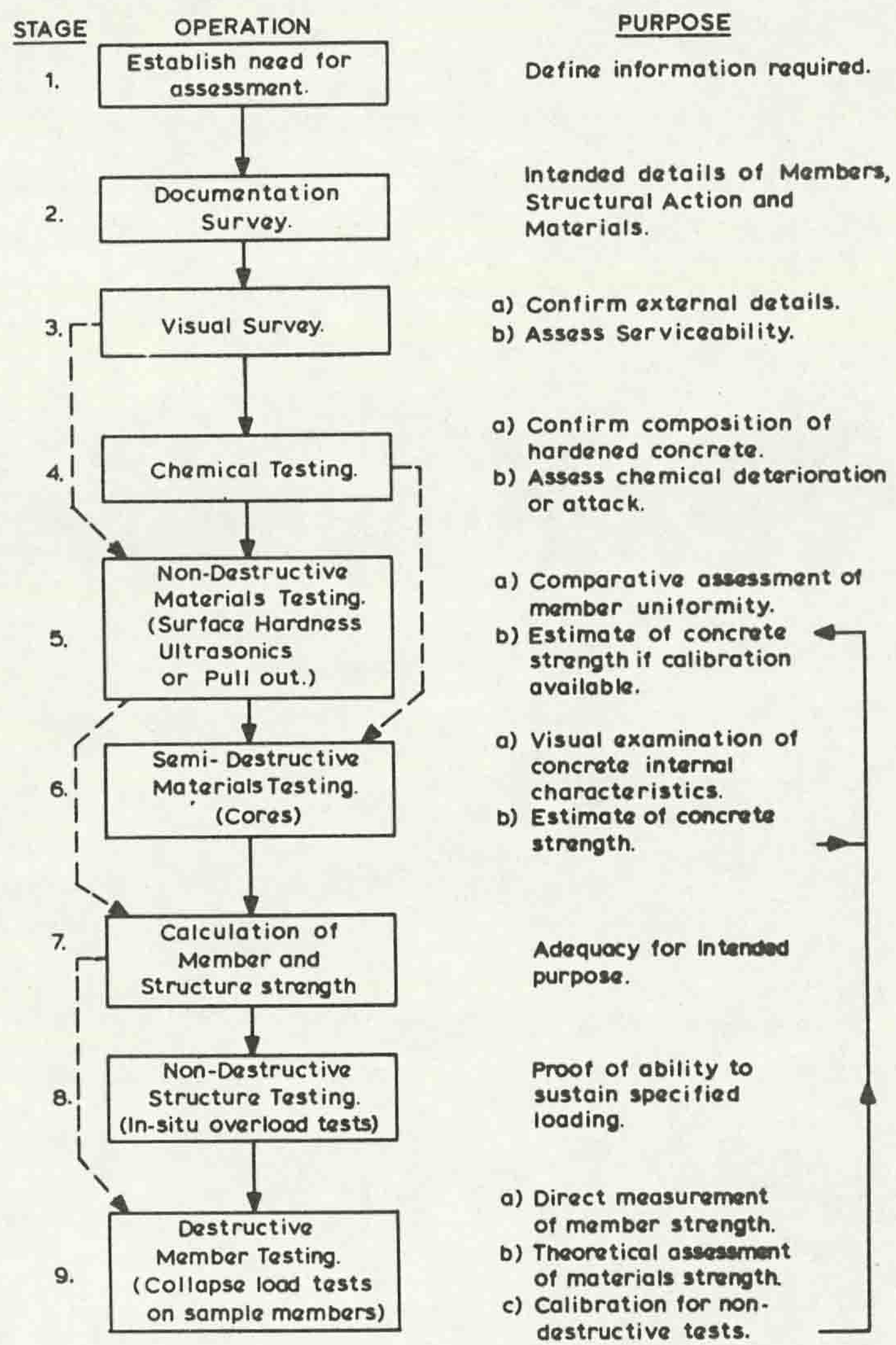
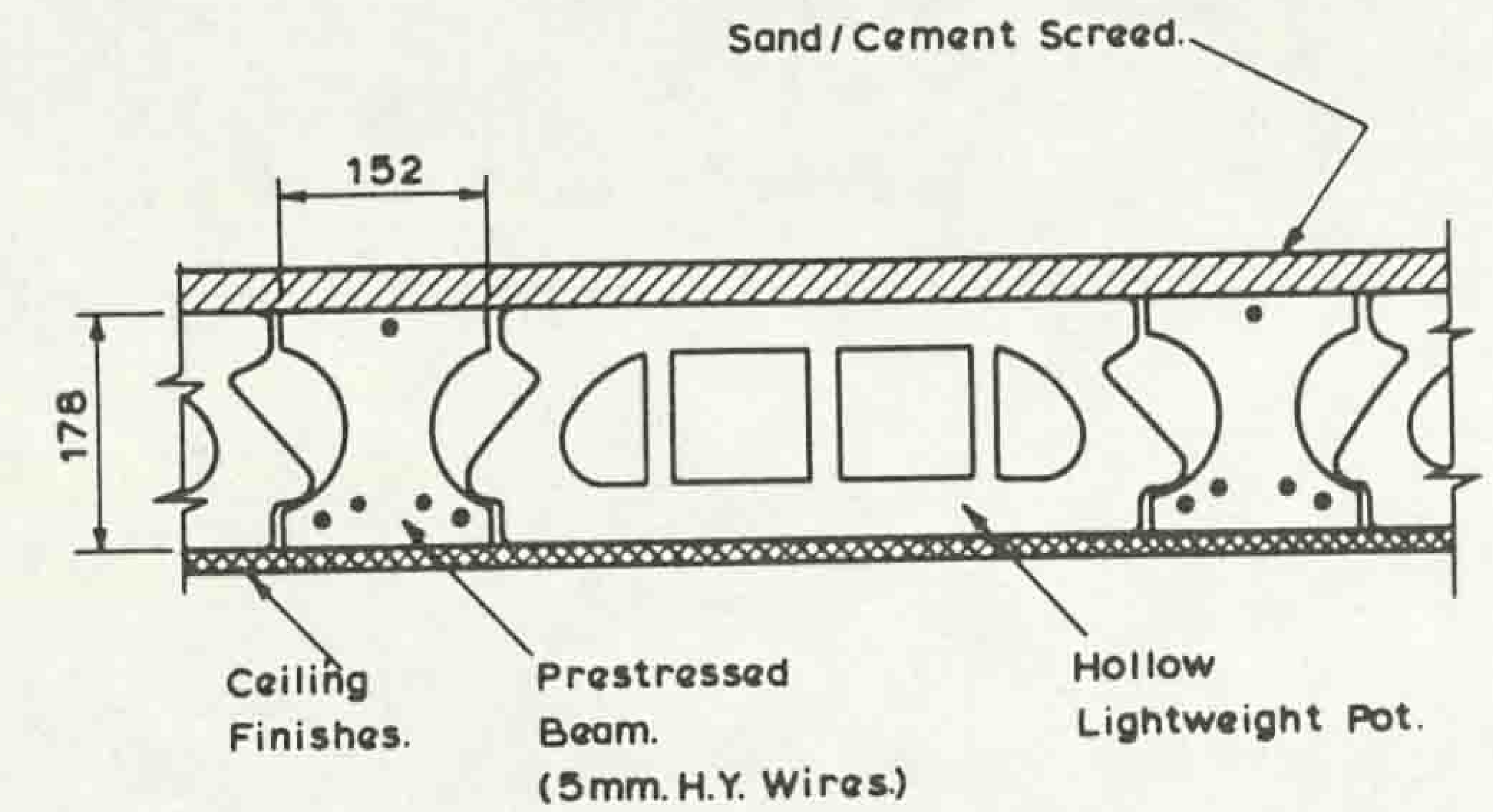
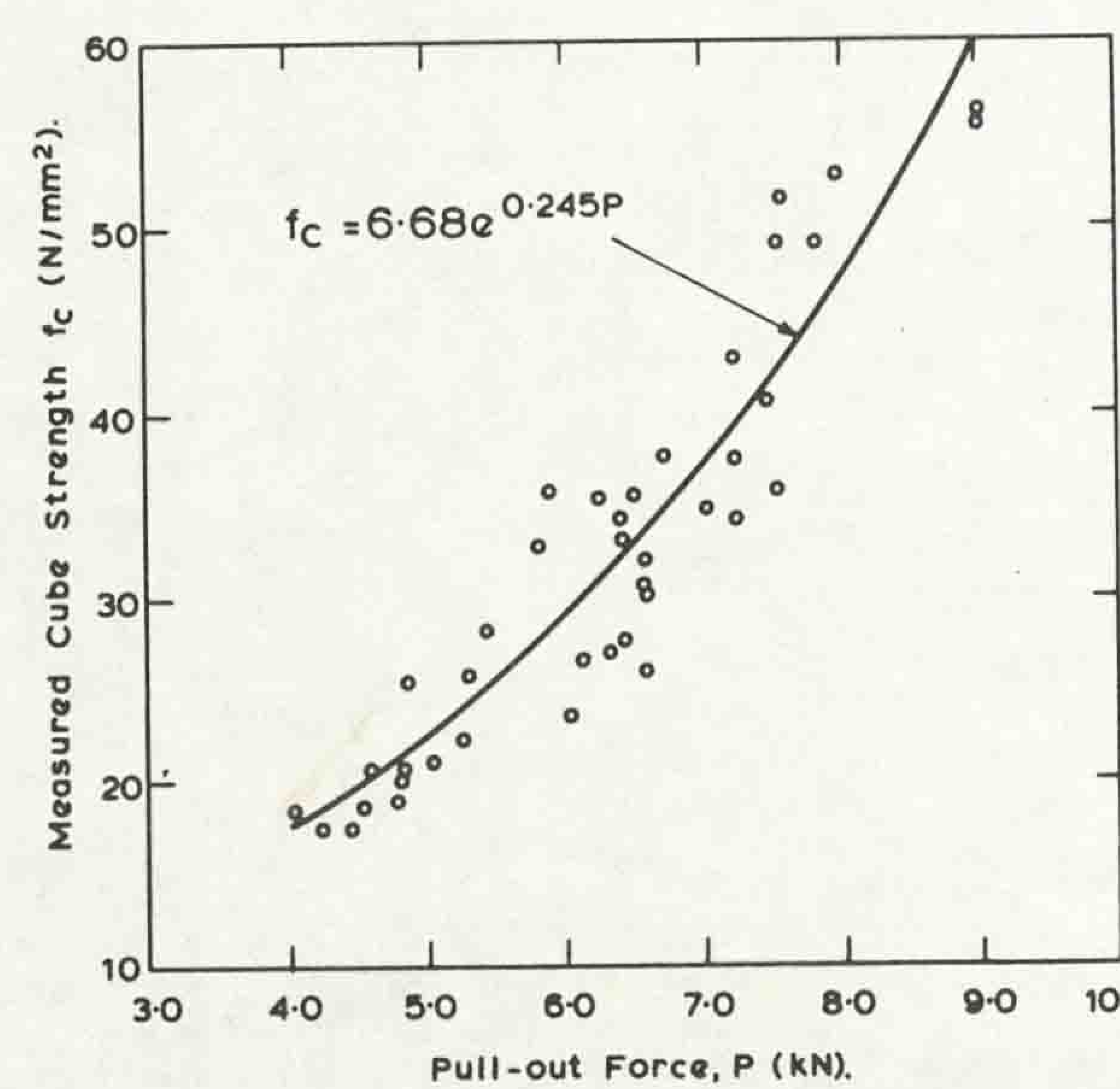


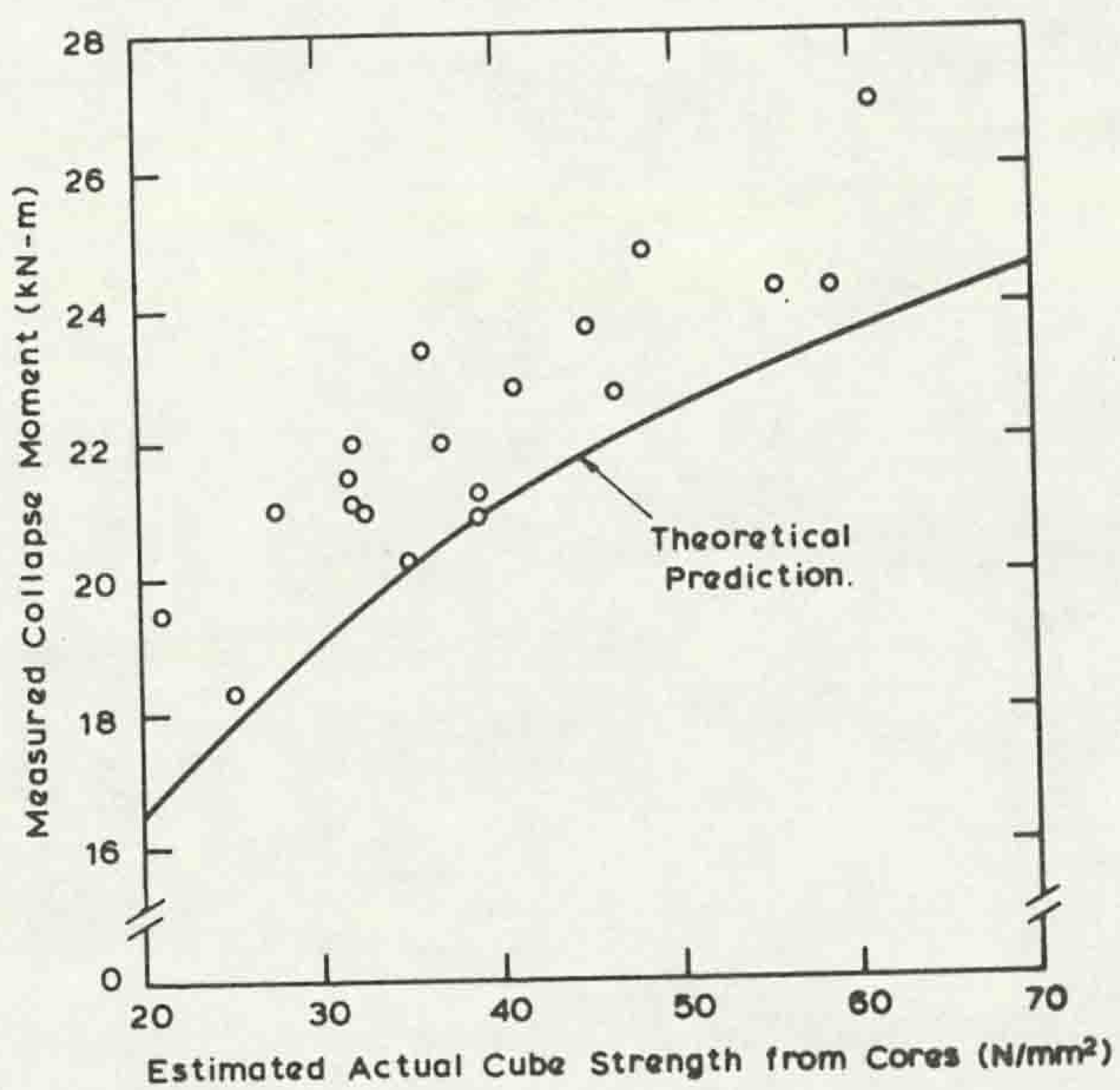
1. Test procedures



2. Typical X-Beam and infill block floor



3. Pull-out calibration - 10mm. gravel and Rapid Hardening Portland Cement



4. Laboratory test results on T-Beams

Paper 18

"Non-destructive Testing -

Planning and Interpretation"

Current Practice Sheet 86

Concrete Vol. 17 No. 8

August 1983 pp.45-46

Current Practice Sheets

Test methods

No 86

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VDC 666.972.017: 620.179.1

Non-destructive testing: Part 1

Planning and interpretation

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Introduction

Non-destructive testing offers significant advantages of speed, cost and lack of damage in comparison with test methods which require the removal of a sample for subsequent examination. These factors will permit more extensive testing and thus enable an investigation to be more comprehensive than would otherwise be possible. The immediate availability of results may also be an important advantage of this type of testing. The range of material properties that can be measured is large, whilst the quality of workmanship and structural integrity may also be checked by the ability to identify and locate voids, cracking, and delamination.

The term 'non-destructive testing' is taken in its broadest sense to include methods which cause localised surface damage but are non-destructive in relation to the body of the concrete under examination. A number of recently developed test methods are of this type and are commonly classified as 'partially destructive' methods. It must be remembered that some other methods which have traditionally been recognised as non-destructive may also cause minor surface marking or staining.

Situations in which non-destructive testing may be valuable

Non-destructive testing may be applied to both new and existing structures. For new structures the principal applications are likely to be for quality control, monitoring of strength development or resolution of doubts about the quality of materials or construction. Testing of existing structures will usually be related to an assessment of structural integrity or adequacy, and material deterioration. In any of these situations localised tests such as isolated cores can be very misleading. Non-destructive tests, however, provide a valuable indication of quality and uniformity, and may also be useful as a preliminary to other forms of testing which are more damaging or expensive.

Non-destructive test methods may be useful for the following purposes:

- Quality control of precast or in situ construction
- Investigation of specification compliance of material supplied
- Examination of workmanship
- Location of cracks, voids, honeycombing, etc
- Monitoring of strength development or long term changes in properties
- Location and assessment of likely condition of reinforcement
- Assessment of concrete uniformity and extending the value of localised destructive tests
- Confirming or locating suspected deterioration of concrete
- Assessment of potential durability
- Providing data for structural capacity or integrity surveys.

Available test methods

These may be grouped into three categories according to the property measured:

- Those which attempt to provide a direct measure of concrete strength
- Those which assess a parameter which can be used comparatively, and in some cases in conjunction with a specific calibration
- Those which directly provide information, other than strength, which may be valuable in its own right.

The range of available test methods is extensive, but the most frequently used techniques are listed in Table 1. Many of these methods have been developed within the past ten years and are summarised in Part 2 of this Current Practice Sheet, with detailed descriptions available elsewhere[1].

Detailed planning of an investigation

The most important stages of an investigation are outlined in Figure 1. It is essential that the first stage must be to establish the reasons for testing, and hence the information that is required (eg strength, uniformity). This will also identify whether information should be related to the surface or interior of the concrete, and whether results are required for specific areas or to reflect general properties. Agreement must be reached prior to

Table 1: Features of principal test methods

Test method	Major applications: Quality Control: Material	Major applications: Quality Control: Workmanship	Major applications: In situ compressive strength estimation	Major applications: Comparative surveys: Strength	Major applications: Comparative surveys: Durability	Test characteristics: Classification	Test characteristics: Type of equipment	Test characteristics: Principal property measured	Test characteristics: Test region: Surface zone	Test characteristics: Test region: Interior
Internal fracture	x		x	x		Partially-destructive	Mechanical	Strength	x	
Pull-out	x		x	x			Mechanical	Strength	x	
Pull-off	x		x	x			Mechanical	Strength	x	
Windsor probe	x		x	x			Mechanical	Strength	x	
Rebound No	x			x	x	Non-destructive	Mechanical	Surface hardness	x	
UPV	x	x		x	x		Electronic	Elastic modulus		x
Pulse echo		x					Electronic	Dynamic response		x
Radiography		x			x		Radioactive	Relative density		x
Initial surface absorbtion		x			x		Hydraulic	Surface absorbtion	x	
Thermography					x		Infra-red detection	Surface temperature	x	
Maturity			x				Thermoelectric or chemical	Maturity	x	x
Cover measurement		x			x		Electro-magnetic	Presence of steel	x	
Radiometry		x			x		Radioactive	Density	x	
Neutron absorbtion					x		Nuclear	Moisture content	x	
Resistivity					x		Electrical	Resistivity	x	
Half-cell potential					x		Electrical	Reinforcement electrode potential	x	

testing between the interested parties on the validity of the proposed procedures, both for testing and interpretation, if later dispute is to be avoided.

Visual inspection also forms an essential component of the preliminary stages of planning, since observation of features such as deflection, cracking and colour as well as practical site circumstances may provide valuable background information influencing the choice of methods and location of test points.

Selection of test methods

A general guide to the suitability of methods for particular requirements is given in Table 1. Important considerations will include:

•*Availability and reliability of calibrations:* These may be necessary in some instances to relate the measured values to the required properties

•*Effect of damage:* This will relate to both the surface appearance of the test member and the likelihood of structural damage resulting from the testing of small section members

•*Practical limitations:* Important features will include the member size and type, surface condition, depth of test zone, location of reinforcement and access to test points. Other factors may also include ease of transport of equipment, effect of environment on test methods and safety of test personnel and the general public during testing

•*Accuracy required:* This will not only influence the choice of test method but also the number of test points

•*Economics:* The value of the work under examination and the cost of delays must be carefully related to the likely cost of a particular test programme.

It will normally be sensible to organise the testing in a sequence, such as that suggested in Figure 1, in which the tests involving least cost and damage are used initially and followed by other methods as necessary. The results of each stage should be analysed in the light of the agreed requirements before proceeding to the next stage. Combinations of test methods may be particularly useful to confirm observed patterns of results or to increase the reliability of estimated properties.

It is important to recognise that some methods are particularly sensitive to variations of testing procedure, whilst in many instances it is only possible to obtain estimates of the required properties by comparative means. Skill and care by the operator will always be necessary and reliable trained staff must be used.

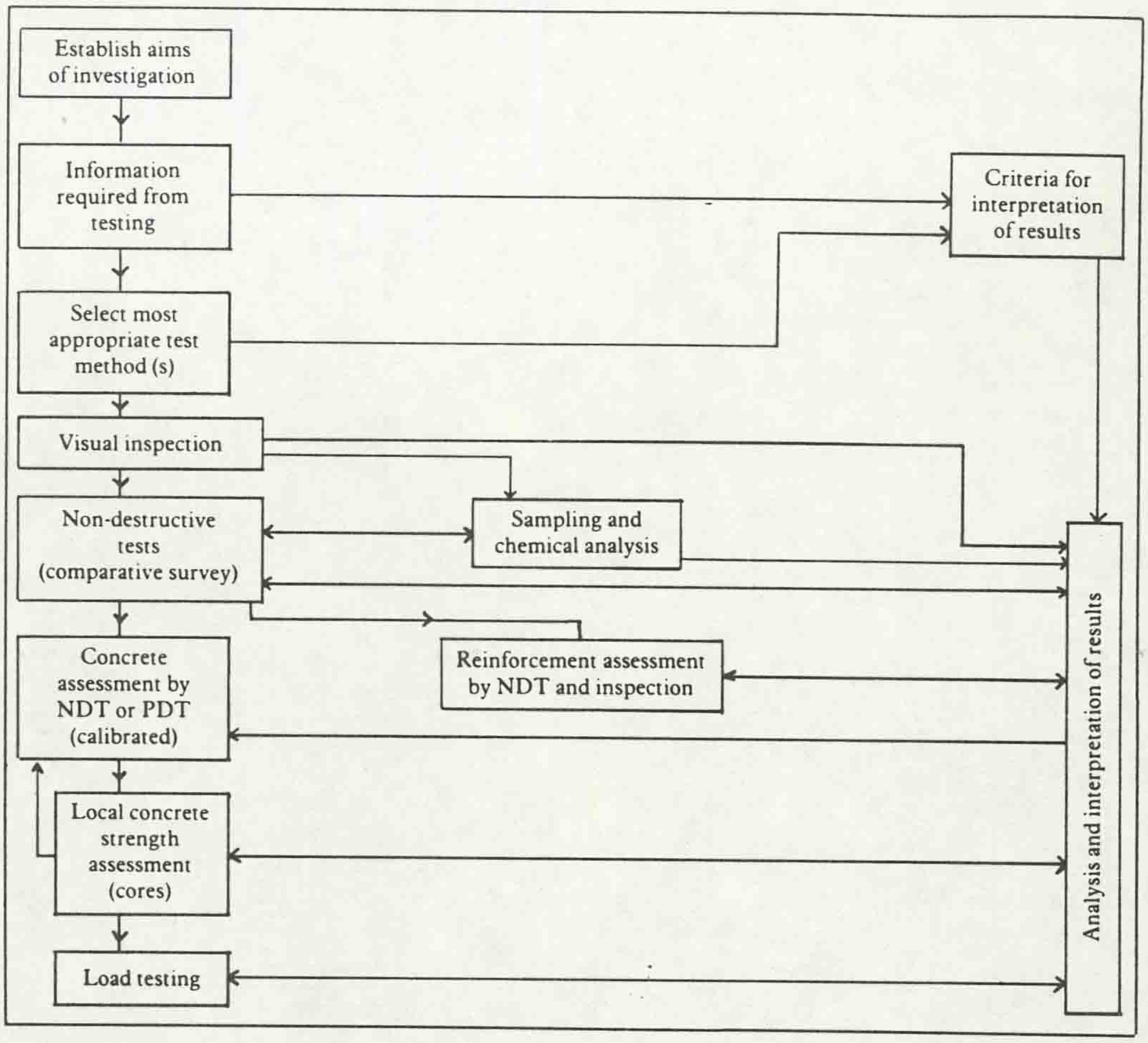

Figure 1: Stages of an investigation

Location and number of tests

The location of test points will be governed by the detailed requirements of the investigation. In some instances specific areas of members will be suspect, possibly as a result of visual examination, whilst in other cases a general comparative survey will be required. If estimates of concrete strength-related properties are required, it is essential to establish the interpretation procedure to be used for relating results either to specification requirements or calculations of structural adequacy. Concrete properties will vary within a member and patterns can be established according to member type. Testing should therefore be concentrated on zones where average conditions of compaction and curing prevail or alternatively in regions where the quality is likely to be lowest[1].

The number of tests required at any given location will depend upon the method used and its 'within-test variability', whilst the number of test locations will be governed largely by consideration of economic factors balanced against accuracy requirements.

Interpretation of results

The nature of non-destructive testing is such that interpretation can only be properly carried out by a suitably experienced engineer. As well as a thorough understanding of the features influencing the results of particular test methods, it is necessary to consider the anticipated variations of properties within members. In some cases it will be possible to obtain information directly from the measured values, assisted by the use of contour plots, histograms, and evaluation of coefficients of variation. In other circumstances the equipment must be calibrated for the particular concrete mix involved.

A major feature of the interpretation of in situ strength test results is the difference between in situ properities and those of standard cube specimens as a result of handling, compaction and curing[1]. This will vary according to member type and is recognised by BS6089: 1982[2] which also suggests that a partial factor of safety of 1.2 should be applied to estimates of in situ strength obtained by indirect means. Comparison with specifications should generally be based on 'average' in situ values related to 'average' standard specimens with due allowance for these differences. Application to calculations should similarly be based on average values, or alternatively on in situ results, from areas of likely minimum strength related to corresponding characteristic values of standard specimens.

Whatever form of non-destructive testing is employed the result will relate only to the points actually tested and any conclusions about the properties of the remainder of the body of concrete under examination must be a matter of engineering judgement. Nevertheless, testing of the concrete in-place may be expected to yield a much more reliable indication of the true properties than tests on standard specimens which relate only to the materials used.

Part 2, to be published in October, will cover developments in test methods and will include references from both parts.

Paper 19

"Concrete strength variations and in-place testing"

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CONCRETE STRENGTH VARIATIONS AND IN-PLACE TESTING

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SUMMARY

Variations of concrete strength within full-scale structural members may be considerable due to variability of supply, compaction and curing. The likely magnitude of these have been assessed for principal member types and it has also been shown that the mean insitu strength can be expected to vary according to member type and may be as low as one half of the strength of a standard wet cured control specimen. The selection of test methods must take account of these factors in conjunction with the purpose of the investigation which will usually be for either performance assessment or specification compliance. The merits of the principal available non-destructive and semi-destructive methods have been compared in relation to these requirements. The anticipated test result variabilities for good construction standards have also been summarised to assist assessment of uniformity. Application of the results of testing frequently requires an understanding of the differences between insitu concrete and control specimens, together with the use of factors of safety to account for the likely variations, and these have been examined for the member types reported.

INTRODUCTION

Small laboratory specimens of concrete can be compacted and cured to a considerable degree of uniformity. As the size of the element increases, however, this becomes more difficult, even under laboratory conditions, and considerable variations of concrete strength will normally exist in full-scale members. This will be especially marked in those cast insitu due to the levels of control which are possible on both material supply and construction procedures. Although the presence of such variations are recognised, their details are generally of little concern to the Designer since factors of safety incorporated into design procedures will allow for their existence. Consequently these anticipated within member variations are frequently overlooked by Engineers, although they may be of considerable importance to the planning and interpretation of in-place testing.

In this paper an attempt is made to quantify such variations for basic member types, and to examine the relationship between strength values measured insitu by non-destructive or semi-destructive testing and values used in specifications and design calculations. Information has been gathered from a variety of published work and combined with the results of the Author's own experimental work to aid a greater understanding of this problem, and a more reasoned approach to the results of in-place strength testing.

CAUSES OF CONCRETE STRENGTH VARIATIONS

These may be attributed to the three basic causes of lack of uniformity of concrete supply, compaction and curing. Concrete supply variations will be due to inevitable differences in materials and batching, which are a function of the degree of control over production, and also due to transport and handling techniques. It is, for example, well known that the strength of concrete from one truck load will vary according to whether it is at the beginning, middle or end of the discharge. Supply variations are obviously not related to member type and may be assumed to be uniformly distributed throughout a structure. They are, however, not easy to measure insitu because of the difficulties of isolating them from compaction and curing differences within members, but may be assessed by consideration of the coefficient of variation of tests taken at comparable locations within a structure or member. Neville (1977) indicates that a value of 14% coefficient of variation on strength of control specimens may be expected for good control, with 18% for fair control. Additionally, Tomsett (1980) suggests that a 16% overall coefficient of variation of measured insitu strength for one load of concrete in one unit may be regarded as satisfactory, or 26% where several loads are involved. These latter values include variations other than just those of the supplied material, but provide a useful guide to the Engineer when interpreting insitu test results.

Compaction and curing differences are closely related to member types and location within the member. Reinforcement will sometimes hinder compaction, but if it is assumed that detailing has been properly executed there will be a general tendency for aggregate to settle and moisture to rise during compaction. Provided that the mix does not segregate this may result in a water/cement ratio differential across the member depth, although the extent of this and the consequent strength range will depend upon the amount and nature of compaction, and member type. The basic aim of curing is to ensure that sufficient water is present to enable hydration to proceed. For mixes with a water/cement ratio below 0.5 self-dessication is possible and ingress of water must be permitted, whilst for higher water/cement ratios the fundamental requirement is that drying out is prevented. Thus mix, member nature, location within member, and curing technique are all variables in this respect, and the extent of possible curing influence is demonstrated by Fig. 1 which is based on the work of Price (1951) for a mix with 0.5 water/cement ratio.

EFFECT OF MEMBER TYPE

Columns

The principal strength differences within columns can be attributed to compaction rather than curing effects. The concrete at lower levels of a column will be compacted by the pressure of that above, and in addition moisture may rise to give an increased water/cement ratio and hence reduced strength in the top zone. Fig. 2, which is developed from results of ultrasonic testing reported by Tomsett (1974), shows a distribution of strength typical of that reported by other investigators.

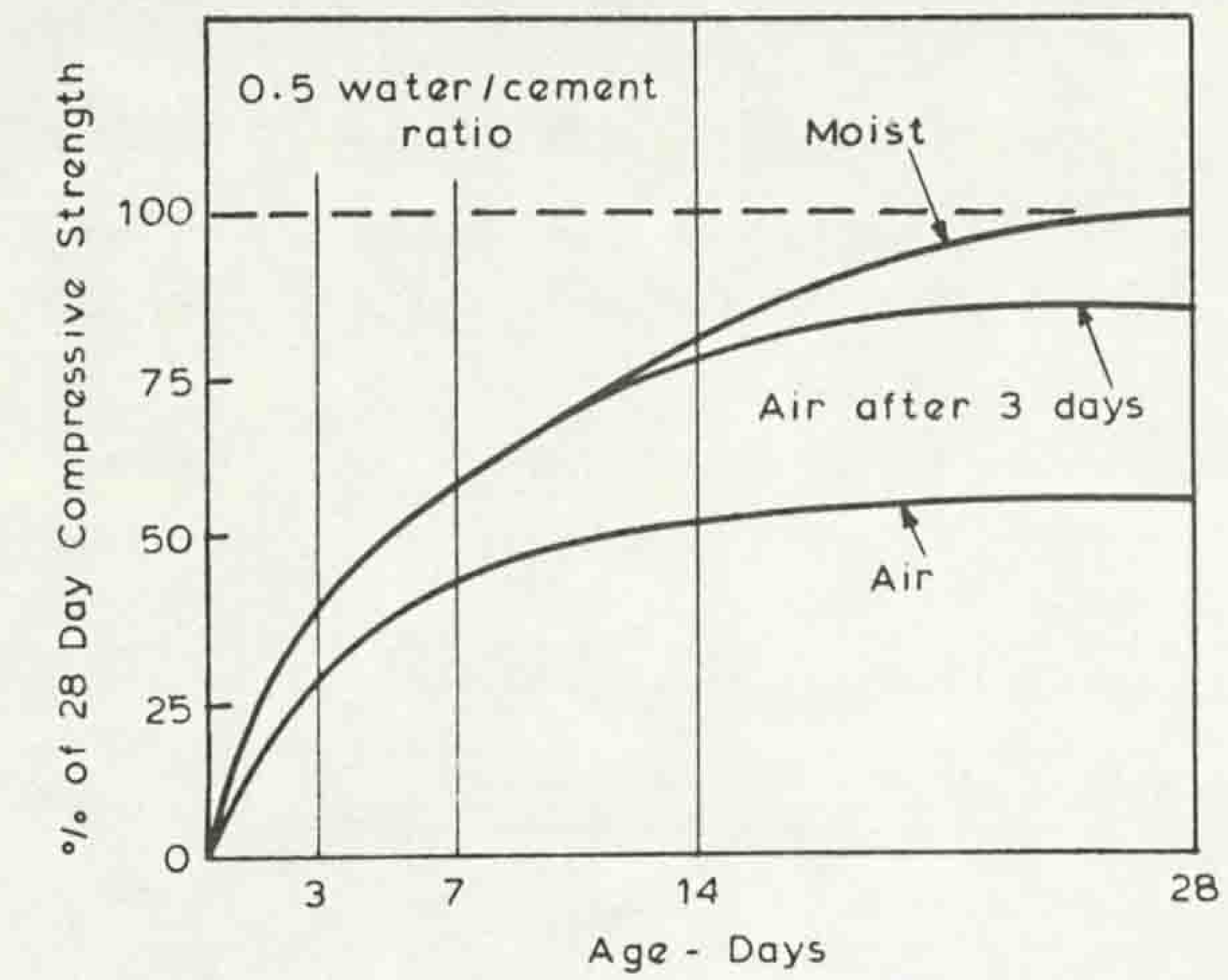

Fig. 1. Influence of Curing on Compressive Strength

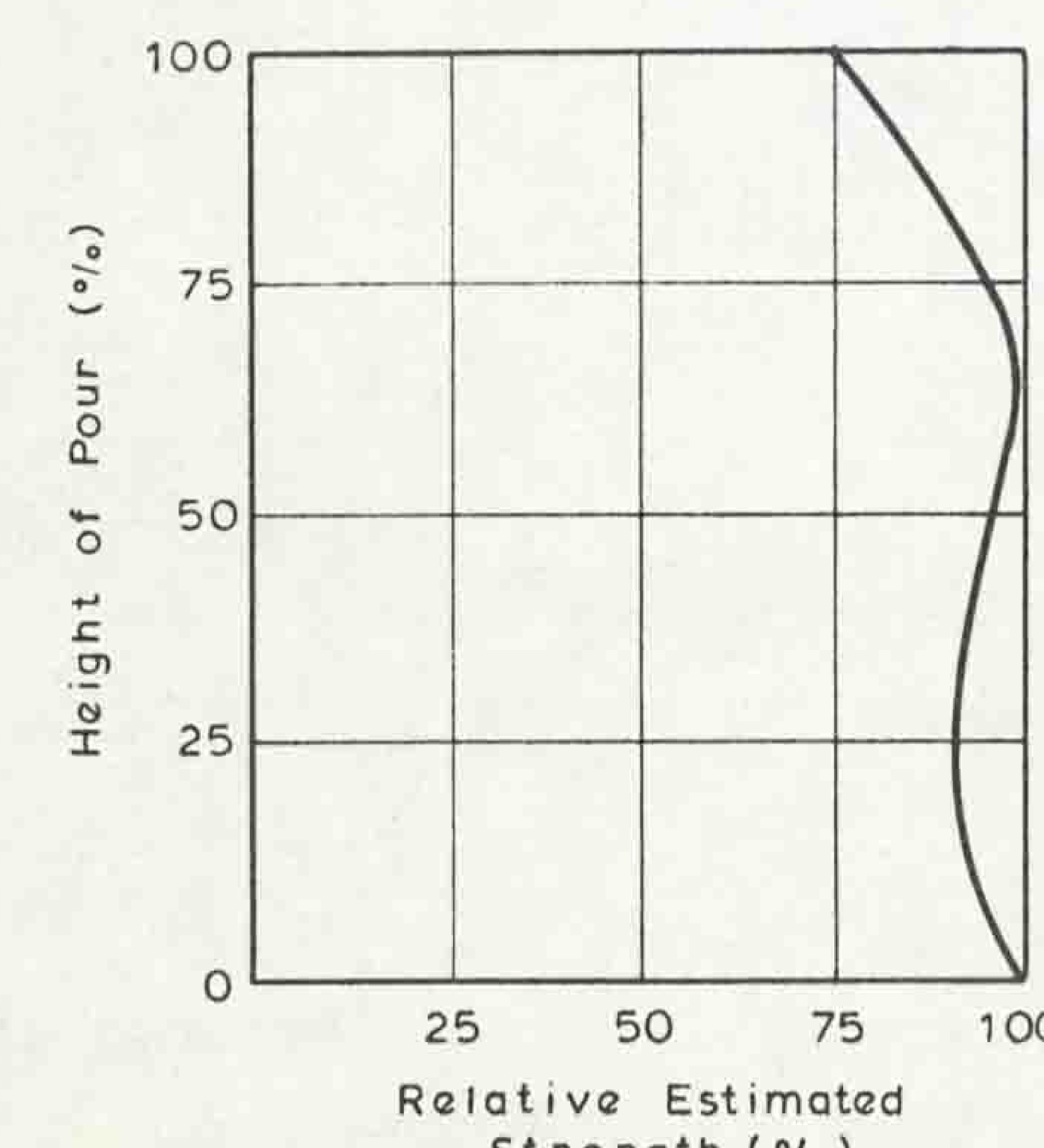

Fig. 2. Typical Column Strength Distribution

It can be seen that the strength is sensibly constant over the lower 85% of the member and that the greatest strength differential is of the order of 20% of the maximum. Davis (1976) has however shown that such variations may not always exist, and also that similar columns may not necessarily exhibit similar strength distribution characteristics. Nevertheless, he reports that for a large number of columns tested over 5 sites the mean strength in the top 500mm. was on average 90% of the mean strength of the remainder. Furthermore, a mean insitu strength of only 65% of the mean standard cube strength is reported.

The above strength estimates have all been based on the results of ultrasonic testing, and thus do not reflect possible strength differences across the width of columns. Curing will not vary with height but exterior surfaces may dry more quickly than the interior and hence develop lower strengths.

Walls

It is to be expected that differentials relative to height will exist, and this is confirmed by Davis (1976) who has shown that in addition there is a tendency for strength to decrease towards the ends of long walls. It would appear that the differential with height is greater than for columns with a more uniform gradient as shown in Fig. 3, and that a reduction of 50% of the maximum may exist.

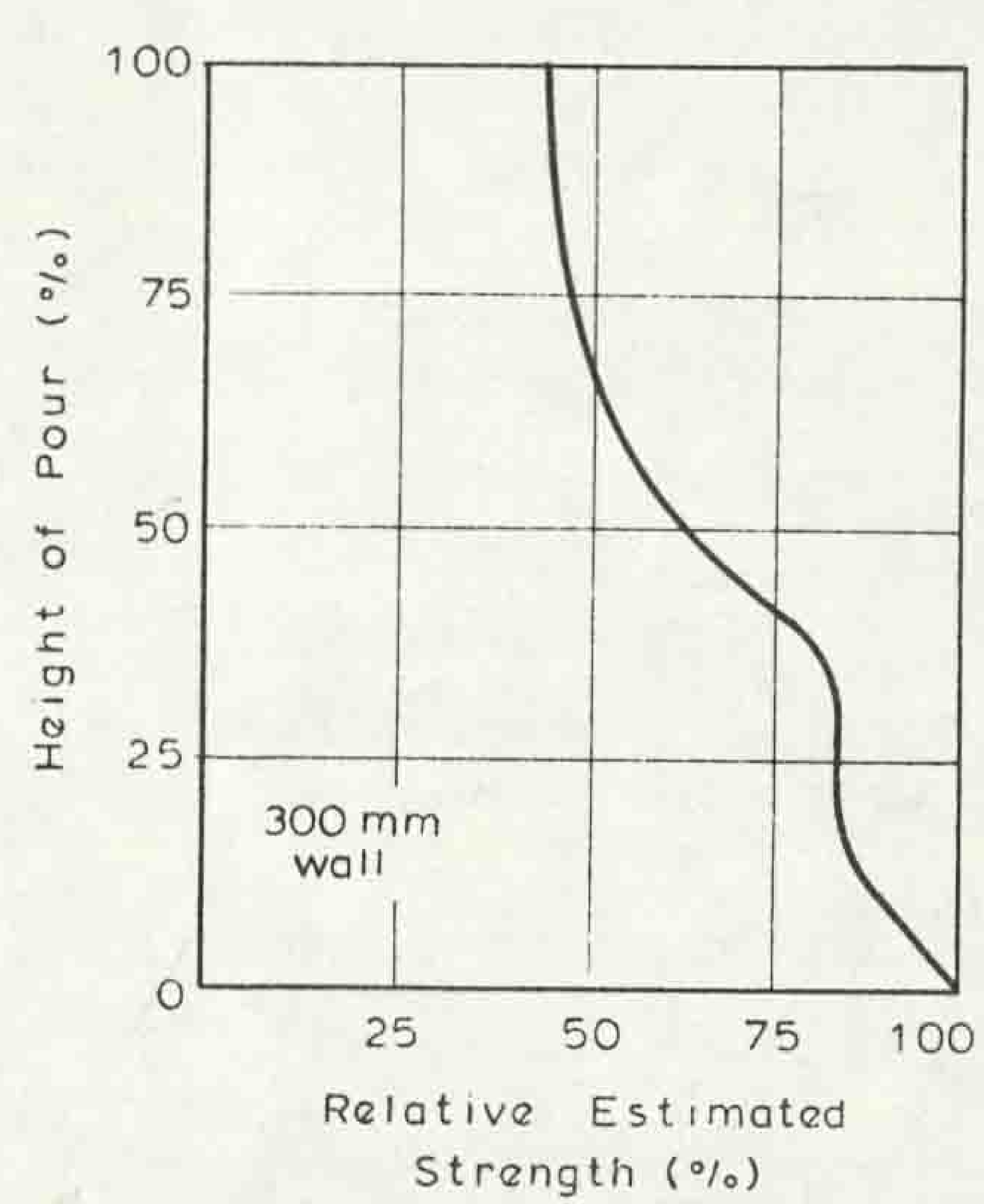

Fig. 3. Typical Wall Strength Distribution

The reasons for this considerably greater strength difference must be due to problems of placing and compaction since curing will normally be similar at all levels of a wall, just as for a column. The observed mean 28 day strength of 64% standard cube is, however, comparable to that reported for columns. Whilst columns and walls will probably receive similar treatment as far as curing is concerned the larger volume of concrete involved in a wall may possibly be more difficult to treat uniformly. It is also interesting to note that the range of strengths is more similar to that reported below for beams, although it is hard to find a convincing explanation for this.

As with the column results, values are based on ultrasonic surveys which do not show up differences between interior and surface zones caused by drying out. Reported results indicating the extent of such differences do not appear to be readily available, but some indication may be given by the results for slabs and beams below.

Slabs

The methods of compaction of slabs can vary considerably and settlement of aggregate may occur if well compacted, whilst if surface tamping only is employed it is possible that lower levels are poorly compacted with a resultant lowering of strength. The dominant feature in relation to strength, however, is likely to be that of curing, since slabs are amongst the most prone of all structural members to drying out. Soffit shutters are likely to remain in place for several days, thus the concrete at the lower levels is only likely to dry by migration of water to the top surface, although self-dessication can be a problem. Unless proper moist curing or membrane sealing is applied to the top surface, this is likely to dry, and the order of possible overall strength reduction is indicated by Fig. 1. The differential across the slab may be influenced by the drying gradient and is likely to increase with member thickness but is unlikely to reach the extent of the 50% reduction at 28 days suggested by the extreme values in that figure. A further feature of slabs is that the surface is often worked by trowelling or some similar technique which will result in a hardening of the surface layer and this must not be overlooked when selecting testing techniques. The measurement of the strength differential is, therefore, not easy since neither ultrasonics or rebound hammer methods may be satisfactorily used for this purpose. However, Lewis (1976) has reported strength differences of up to 18% across 300mm. slabs on the basis of core results, although a gradient was not always found to exist. A recent insitu survey by the Author on a 12 year old 1.5m. deep voided bridge deck slab using the Windsor Probe method, as described by Bungey (1981), indicated top surface strengths which were 11% weaker on average than soffit values with a maximum measured differential of 28%. It has also been suggested by a Concrete Society Technical Report (1976) that curing influences will only affect a surface zone of 50mm. thickness but that the strength reduction may be up to 25% due to this cause.

Whilst it would appear that strength differences across slabs are less than for walls, the mean strength may be considerably less and Davis (1976) reports an average value of 45% Standard Cube strength at 28 days for floor slabs of up to 300mm. thickness. For the thicker slab examined by the Author, and described above, it was not possible to assess this ratio precisely but a value in the range 50-70% was estimated on the basis of the available information concerning the mix used.

BEAMS

Published data on beams is limited, thus an investigation has been carried out by the Author in which 2 No. 4.5m. long, 0.5m. deep reinforced concrete beams were constructed in the laboratory. Each was manufactured from at least 7 individual batches of 10mm. gravel aggregate mixes using Portland Cement, using a $0.1m^3$ pan mixer and with compaction provided by immersed poker vibrators. The beams were cured under moist hessian for 7 days, and subsequently exposed to the laboratory atmosphere of 20°C and 55% relative humidity, whilst 150mm. test cubes were wet cured at 20°C until testing. The beams were allowed to dry for several months prior to test, and were then examined by ultrasonics, 44mm. cores, and internal fracture testing.

An ultrasonic calibration with strength was obtained for the mix by tests on dry cubes at varying ages, and this used to produce relative strength 'contours' of which that shown in Fig. 4 is typical. Numerical analysis of the pulse velocity results is also incorporated into Table 1.

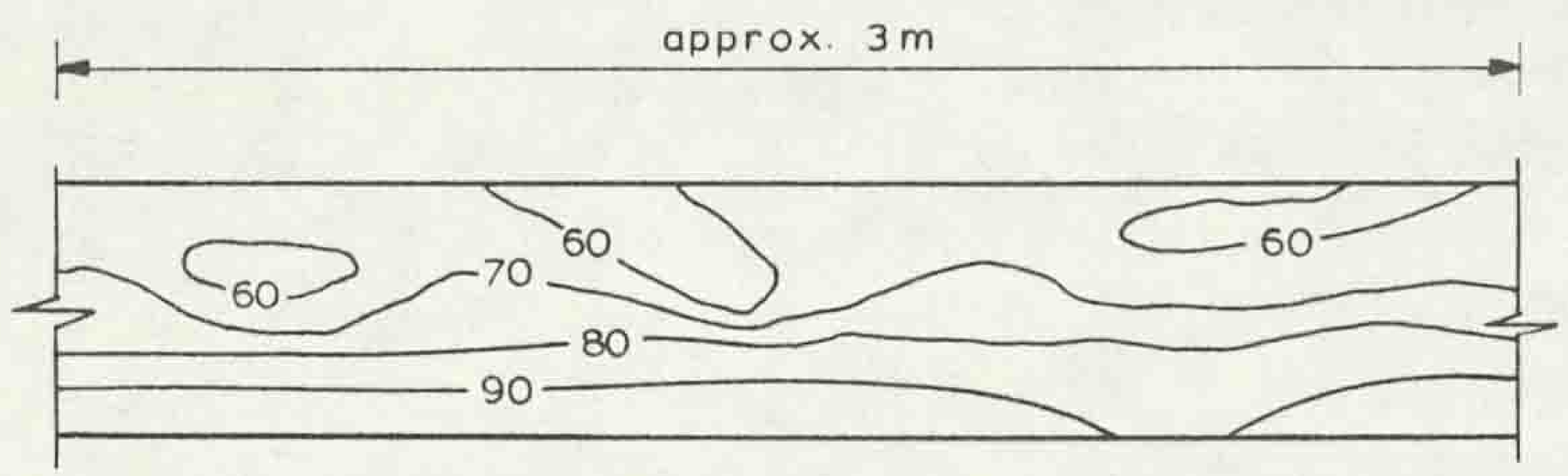

Fig. 4. Relative Percentage Strength Contours for Beam

The cores were cut at 3 levels of the beams and tested wet in accordance with BS.1881, Pt. 4 (1970). Each core passed completely through the beam and was split into 3 specimens prior to test and assessment on the basis of earlier work by Bungey (1979). A total of 81 cores from beam 1 and 90 from beam 2 were tested and the results are summarised in Table 1. Comparison of the exterior and interior portions of the cores also revealed a fairly consistent strength ratio in the range of 90-95%, with the interior being the stronger.

Internal fracture tests, in which a 6mm. bolt is pulled from a 20mm. deep hole drilled into the concrete surface as described by Chabowski (1977), were also taken at similar levels of the beams. A total of 48 such tests were made for each beam and yielded the estimated strengths also shown in Table 1.

It is apparent from the summarised results that there is an approximately linear strength gradient across the depth of the beams, with an average top zone strength which is approximately 30% lower than that in the bottom regions of the member. This trend is reflected by each of the testing methods employed.

Method		Ultrasonic Pulse Velocity			Internal Fracture			Cores			
		Strength (N/mm^2)	Coefft. of Variation	% Strength Reduction	Strength (N/mm^2)	Coefft. of Variation	% Strength Reduction	Strength (N/mm^2)	Coefft. of Variation	% Strength Reduction	Average % Reduction
Beam 1	Top	22	6	32	22	16	39	23	5	15	29
	Mid	25	8	22	28	15	22	22	5	18	20
	Bottom	32	6	0	36	9	0	27	5	0	0
Average		26			29			24			
Beam 2	Top	21	8	34	22	13	37	19	6	27	33
	Mid	29	8	9	27	14	23	23	6	12	15
	Bottom	32	6	0	35	11	0	26	8	0	0
Average		27			28			23			

Table 1. Estimated Cube Strengths Based on Non-Destructive Testing

The measured strengths of the standard wet cured and wet tested 150mm. cubes were 32N/mm^2 at 28 days and 36N/mm^2 at the age of beam testing in both cases. Thus it can also be seen that the average insitu strength is considerably lower than that of the standard control cubes. The estimated strengths based on Ultrasonics and Internal fracture tests relate to dry concrete, whilst cores relate to wet specimens and may thus be expected to be lower. The average difference of 15% between wet and dry results agrees closely with that normally anticipated. It can thus be taken that the average actual insitu dry strength of the concrete is 27.5N/mm^2 in both cases, this being 86% of the standard 28 day strength. The minimum insitu strength is, however, as low as 60% of the 28 day value locally, or if these results are expressed in terms of the equivalent wet cube strength a mean of 73% and minimum of 50% are obtained. The coefficients of variation for concrete at similar levels are generally consistent and at a value to be expected for laboratory produced concrete, although internal fracture values reflect a greater test variability. Overall values for each beam are summarised in Table 2 from which it can be seen that the compaction and curing differences produce results which reflect a good construction standard on the basis of Tomsett's (1980) recommendations.

Table 2. Summary of Overall Strength Coefficients of Variation for Beams

Beam	Test Method		
	Ultrasonic Pulse Velocity	Internal Fracture	Cores
1	16%	24%	11%
2	21%	23%	15%

CHOICE OF TEST METHOD AND LOCATION

It will be apparent that the location of tests must be given careful consideration, and that the requirements for the results may influence the choice of method. It is essential that the Engineer is clear in his mind concerning the aims of any in-place testing, which will normally relate to either specification compliance or performance assessment. Specification compliance will normally be based on mean values whilst for performance assessment or investigation of deterioration minimum strength values in critical locations will be of greater importance. This must be considered in relation to the physical nature of the member under test, together with an understanding of the limitations of the various test methods available.

Rebound Hammer

The use of this well-established technique for surface hardness testing is largely restricted to testing of a specification compliance nature at ages of up to 3 months. Although only one exposed surface is necessary, surface skin effects dominate and results offer little indication of the quality of the interior of a member at greater ages.

Windsor Probe

This method, in which a hardened steel alloy bolt, or probe, is fired into the concrete by a driver using a standardised powder cartridge has been gaining in popularity in the USA in recent years. The depth of penetration, which may be up to 50mm. is measured, and it has been established experimentally by Malhotra (1976) and others that linear correlations with strength can be obtained with aggregate properties as the principal variable. Despite greater cost the method has many advantages over rebound hammers since it is concrete between 25 and 75mm. below the surface that influences the penetration. Thus although still a surface zone test, a surface skin or texture will not prevent its usage, and by appropriate choice of test locations the method may be used for either of the two major types of assessment for concrete of any age. Calibration for the specific aggregate may be necessary where an absolute strength estimate is required, but for comparative work the test has considerable potential although a minimum edge distance requirement of 150mm. precludes usage for small members.

Pull-out

This takes the form of measuring the force required to extract an insert which may either be cast into the surface as described by Sorensen (1973) or fixed into a drilled hole as described by Chabowski (1980) as the Internal Fracture Test. In either case, the depth of the test is less than 25mm. below the surface and the method is thus another form of surface zone testing. It would appear that skin effects are insignificant and that specific calibration is less important than for the other tests described. The major disadvantages, however, are that of pre-planned usage where cast-in inserts are involved, and considerable scatter of results where drilled holes are used with expanding wedge anchor bolts. Application of cast-in inserts will be restricted to compliance testing, and this is gaining some popularity both in Scandanavia and in the United States where there is a trend towards in-place testing in lieu of standard control specimens. Internal fracture tests, however, are only likely to be used where Windsor Probes or Cores are inappropriate.

Ultrasonic Pulse Velocity

Without doubt this is the most efficient method of comparative examination of the interior of a member, although absolute strength correlation is subject to many well-established variables. The principal limitation to reliable usage is the need for two opposite faces of the member to be accessible and to be of a smooth texture. In such cases the method may be used in conjunction with appropriate calibration charts for whatever type of survey is required. The speed and truly non-destructive nature of this test means that it is especially suitable for situations in which large areas or volumes of concrete must be surveyed in relation to constructional uniformity and quality.

Cores

Cores of more than 100mm. diameter are by far the most reliable method of determining insitu concrete strength at the point at which it is required. They are, however, expensive and often disruptive to drill, and frequently little thought is given to their location, proper testing, and interpretation. It is very seldom that sufficient cores are taken to permit a realistic assessment of the concrete, and this is particularly true where member size necessitates the use of smaller diameters. Bungey (1979) has shown that the reliability of small cores is considerably less than for larger specimens, whilst Lewis (1980) has indicated the importance of considering core size in relation to location at the interpretation stage. This is especially true for slabs where small cores from the top surface may give different results to larger cores, which will reflect the concrete strength at a lower level. A similar situation may also be expected to hold for other member types in view of the differential between inner and outer zones of beams reported above.

INTERPRETATION OF TEST RESULTS

It is apparent from the beam test results given in Table 1 that estimated strengths vary considerably according to location and test method. The reasons for this have been discussed previously and must be taken into careful account if insitu testing results are to be of any value for comparison with specifications or calculations. In situations where the purpose of testing is to assess the control of construction that has been achieved it will normally be necessary to consider the variability of test results. This is best achieved by examination of the coefficient of variation, and anticipated test result values for good construction are suggested in Table 3. These relate to a single site-made member consisting of a number of batches.

Table 3. Suggested Coefficients of Variation for Various Test Methods Relating to Good Construction Quality

	Test Method				
	Ultrasonic Pulse Velocity	Cores		Internal Fracture	Windsor Probe
		'Large'	'Small'		
Coefft. of Variation	2.5%	10%	15%	20%	4%

Where the results of insitu testing are to be used to provide a strength estimate great care must be taken that calibrations are based on control specimens of comparable size to those upon which specifications and calculations are based. For example, laboratory calibrations may relate to 100mm. cubes whilst 150mm. is the standard size assumed in specifications. Although the strength difference is only likely to be about 5%, this may be added to a further error due to lack of comparability of moisture conditions at the time of test between insitu concrete and the standard control specimens which are generally assumed to be saturated. Comparison of wet cured and wet tested laboratory specimens with dry cured and dry tested specimens of the same concrete yielded a dry strength between 5 and 10% greater than the wet strength at 28 days, but in view of the reduced hydration likely in the dry specimens the effect of moisture condition at time of test is likely to exceed this value. Neville (1977) proposes a value of 10% measured strength difference due to this cause, whilst other investigators suggest that it may be as high as 15%. It is therefore essential that the moisture conditions under which calibrations are obtained is known and taken into

account when considering strength estimates, as illustrated by the beam tests described above.

Specifications are generally based on 'minimum' or 'characteristic' concrete strengths, whilst testing produces an average existing value. For compliance purposes this should be compared with the appropriate production target mean value with due allowance for the anticipated insitu reduction according to member type. Thus, measured average values for thin slabs could justifiably be increased by up to 2.2 times, beams and walls by 1.5 times and columns by 1.18 times for the purpose of such a comparison. Alternatively, the minimum measured values can be compared with the required characteristic value divided by the appropriate ultimate factor of safety which is intended to account for such variations, and would be taken as 1.5 irrespective of member type in the U.K. It will thus be seen that the adequacy of concrete in slabs in practice may depend upon an 'undesigned' margin of concrete production above the specified requirements, coupled with strength enhancement due to dryness.

Whilst calculations are similarly based on minimum control specimen values it may be justified, except where saturation is possible, to use measured dry insitu values in assessments of existing structures but with the materials factor of safety eliminated or reduced. The factor used must depend upon the refinement of the testing in terms of extent and location relative to critical areas.

CONCLUSIONS

The variations of concrete strength that may occur within a member can be considerable, especially in walls and beams, and insitu testing where method and location are based solely on convenience of access or operation may result in serious errors of results in relation to their intended use or applications. Anticipated within member differentials must be considered at the planning stage and interior/exterior strength differences must not be overlooked. An assessment of coefficients of variation of test results may also provide a useful guide to the quality of construction provided that a sufficiently large number of tests are made and that the variability associated with the test method is taken into account. The average insitu strength of concrete may also be substantially less than that of control specimens and will vary according to member type, with slabs likely to be the most critical. Account must be taken of this when determining specification compliance and particular attention must also be paid to the relative moisture conditions of the insitu concrete and control specimens.

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Paper 20

"Assessment of Reinforced Concrete
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(Co-authors: R. J. Cope and P. V. Rao)

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ASSESSMENT OF REINFORCED CONCRETE BRIDGE SLABS

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SUMMARY

The integrity of a reinforced concrete bridge slab may need to be assessed when the structure is showing signs of possible distress, excessive cracking or deflections; when an overload is expected; and when structural modifications are desired. Methods of estimating concrete strength in prototype structures are described, and interpretation of such measurements for use in calculations is discussed. Use of non-linear finite element procedures to predict the complex stress state compatible with measured deflections is described. Methods for extending the analysis to estimate the effects of overloading and structural modifications are proposed.

INTRODUCTION

Assessment of a reinforced concrete bridge slab is required when there is visual indication of distress, when a particularly heavy loading is to be carried, and when structural modifications are contemplated.

Bridge slabs built to current standards are unlikely to exhibit excessive cracking and displacement due to gravity loading, but some skewed and curved structures that were designed before there was a proper understanding of the factors affecting their stiffness and strength have given rise to concern (see, for example, New Civil Engineer, 1981).

Some top slabs of integral beam and slab bridges have shown distress that has led to spalling and corrosion of steel (see, for example, Maeda et al, 1980; Csagoly et al, 1980). Interestingly, assessment of such structures by field tests has indicated that response to concentrated loading is closer to that predicted by non-linear analysis, than by the linear plate theory commonly used in design.

Assessment for overload conditions is likely to increase in importance as there is pressure in many countries to permit heavier axle loads and increased traffic flow is an almost universal phenomenon.

Field testing can be broadly classified under material testing and load testing for structural response. Load testing is immensely valuable for assessing overall structural behaviour and for assessing assumptions made in design. However, as it provides no detailed information on material properties and is expensive, it tends to be used only as a last resort. In this paper, attention is concentrated on cheaper and less disruptive methods non-destructive testing for assessing concrete strengths The likely variations of material properties over plan and depth of a bridge structure are described. The relationship between site measurements, equivalent standard specimen and design strengths are discussed.

For evaluating the response of existing structures, and for better understanding of structural behaviour, in addition to field testing, relevant analytical methods are needed. Since non-linear finite element techniques have the capability to utilise data from 'as built' drawings and non-destructive testing they seem to offer a suitable method for development. Once the finite element material model is properly calibrated, its predictive ability can be used to check alternative modes of alleviating distress and response to overloading. By providing insight into structural behaviour they could also lead to improved design procedures.

IN-SITU MEASUREMENT OF CONCRETE STRENGTH

In-situ measurement of concrete strength for use in calculations is not an easy task. The difficulties arise in three ways:

(a) Practical application of test methods.
(b) Interpretation of results in the light of test calibrations and in-situ strength variability.
(c) Interpretation of results for use in calculations.

Although these factors are inter-related, it is convenient to discuss them under separate headings.

Practical Application of Test Methods

The various non-destructive tests which are available have been widely described in technical literature. The principal methods of use for slabs are Rebound Hammer, Ultrasonic Pulse Velocity, Windsor Probe, Internal Fracture and Cores. Table I indicates the principal features of these methods, relative to each other, assuming good access and test conditions.

It has become apparent, however, from investigations of a number of highway bridge deck slabs that good test conditions are seldom available. Top surfaces generally have surfacing which must be removed if tests involve access to this surface, and even after removal the remaining concrete surface is seldom in a suitable condition for application of rebound hammer or ultrasonic pulse velocity methods without preparatory work such as grinding down. This negates the chief advantages of these methods which are speed and low cost enabling a comprehensive survey involving many readings. It has also been found that where surfacing patches are removed these areas trap surface water and hence give moisture conditions which are unrepresentative of the deck as a whole, and can cause calibration problems for rebound hammer and ultrasonic methods. Although rebound hammer readings could be taken on the soffit, which usually has a reasonably smooth surface, these should not be used for strength estimation where the concrete is more than three months old due to the possibility of carbonation effects.

Method	Cost	Time	Damage	Zone Assessed	Reliability of Strength Calibration
Rebound Hammer	Low	Fast	None	Surface	Poor
Ultra-sonic	Low	Fast	None	Good Average	Mod.
Windsor Probe	Mod.	Fast	Minor	Near Surface	Mod.
Pull-out	Mod.	Fast	Minor	Near Surface	Mod.
Cores	High	Slow	Mod.	Any	Good

(Mod. ≡ Moderate)

Table I - Relative Features of In-situ Test Methods

It is seldom practicable to take ultrasonic pulse velocity readings on the soffit only due to flexural and shrinkage cracks which are usually present. Although direct measurements through the depth of the member may be possible with suitable top surface preparation, it has been found that reliable results cannot be obtained in regions of visible cracking. This is due to the effects of internal cracks on both path length and width. In regions where reliable readings are possible it is essential that absolute strength estimation is based on a calibration for the particular mix.

Windsor probe and pull-out (or internal fracture) tests do not require the same surface preparation, but surfacing must be removed. Both tests relate to concrete near the surface, and leave localised damage to be made good. Whilst pull-out tests can yield an estimate of strength on the basis of a generalised calibration curve the accuracy is poor. Windsor probe testing requires a calibration for the appropriate aggregate type and hardness, and provided this is available, is an easier and more reliable method to perform under site conditions. The method has been successfully used by the Authors on bridge decks whilst in service, and a test rate of four locations per hour has been found reasonable for one operator working on a deck soffit using a small mobile platform. Readings on a prepared top surface could be made at up to twice that rate.

Cores offer the most reliable method of concrete strength determination, but are expensive, slow and often difficult to obtain. Consequently, the number of test locations will be limited since groups of at least 3 cores are required at each location to obtain a reliable strength estimate. Special care must be taken to follow the recommendations of an appropriate 'standard' concerning cutting, capping and testing. This will normally require a minimum diameter of 100mm, which should be possible with most slabs and with care, an estimate of strength at a particular depth of slab may be possible. The temptation to economise by using smaller diameter specimens should be resisted, since the accuracy will be considerably reduced.

A combination of Windsor probe and core testing has been found to have advantages when a limited number of cores can be used to confirm calibration of the Windsor probe which is used for a more widespread survey. For both methods, reinforcement should be located by covermeter and avoided if possible.

Interpretation of Results

Firstly, the results of individual tests must be assessed to provide strength estimates at particular locations, together with the likely accuracy of those estimates. When that has been done, the variability of estimated strengths over the slab can be assessed, and a view formed of the representativeness of the results in relation to the whole deck.

Under ideal conditions of test and availability of calibrations the accuracies of strength prediction at a particular in-situ location are unlikely to exceed the values given in Table II (Bungey, 1981). As mentioned above, ideal conditions seldom exist for prototype structures, and estimates are, therefore, likely to be of even lower accuracies.

Method	Maximum Likely Strength Accuracy (95% Confidence Limit)
Ultrasonic	±20%
Windsor Probe	±20%
Pull-out	±30%
Cores - large ≥100mm	± 6%
(4) - small	±18%

Table II - Strength Accuracies Under Ideal Conditions

It is well established that considerable variations in in-situ concrete strength are to be anticipated due to compaction, curing and material supply variability. Variations in plan for deck slabs are primarily likely to be due to supply and compaction effects and are, therefore, of a random nature. A coefficient of variation of 15-20% on concrete strength has been suggested for average quality construction (Mirza, 1979), whilst for excellent construction standards a 10% coefficient of variation is to be expected for typical structural quality concrete. These figures are based on site made, laboratory cured control specimens, with a low testing variability. Greater variations may be expected for the same concrete on site.

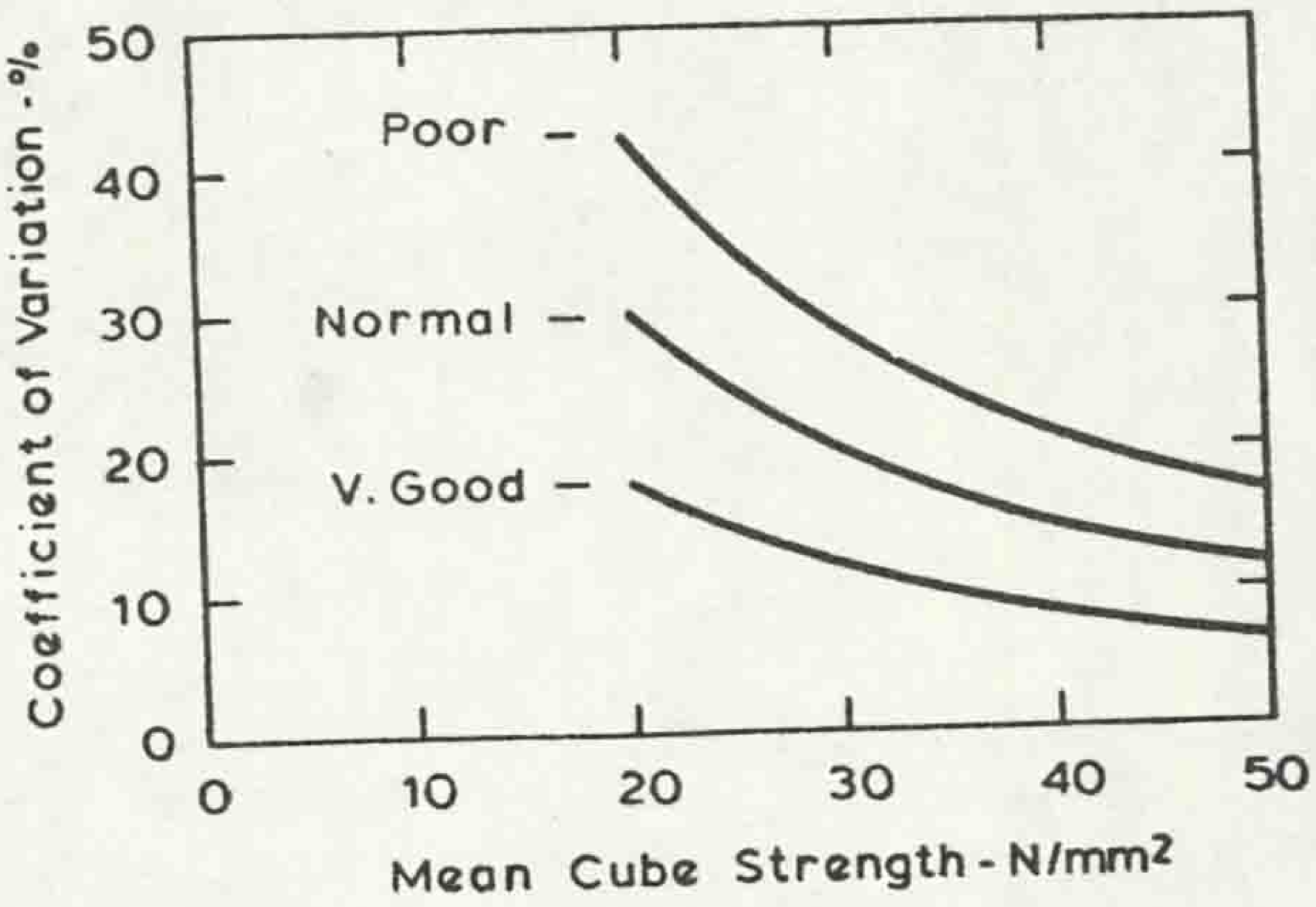

Fig. 1 -- Variability of In-situ Concrete Strength measurements relative to construction quality

It is well known that for a particular level of construction quality, the coefficient of variation is a function of the mean concrete strength. Figure 1 suggests likely values of coefficient of variation for mean in-situ strength measurements relating to three levels of construction quality based on a variety of European and North American sources. From this figure it can be deduced that for a typical bridge concrete of 40N/mm² mean strength, for example, a standard deviation of 6N/mm² is likely for normal quality construction. This corresponds to 95% confidence limits of ±10N/mm² about the mean value. The use of strength estimates based on mean in-situ test values must make due allowance for this, as well as for the accuracy of the test itself.

Variations in strength through depth of well compacted bridge slabs are predominately due to curing effects. It has been demonstrated that for 500mm. deep beams a considerable strength reduction may be expected to exist from bottom to top, even when construction is under laboratory conditions (Bungey, 1980). Similar strength differentials are anticipated in slabs of this and greater thicknesses. The published evidence available for thin slabs of 100-200mm. thickness, however, suggests a smaller strength reduction which is concentrated near to the top surface. These features are summarised in Figure 2. A further aspect with thin slabs is that trowelling of the surface can produce a hardening of the surface layer and this can affect some test results unduly.

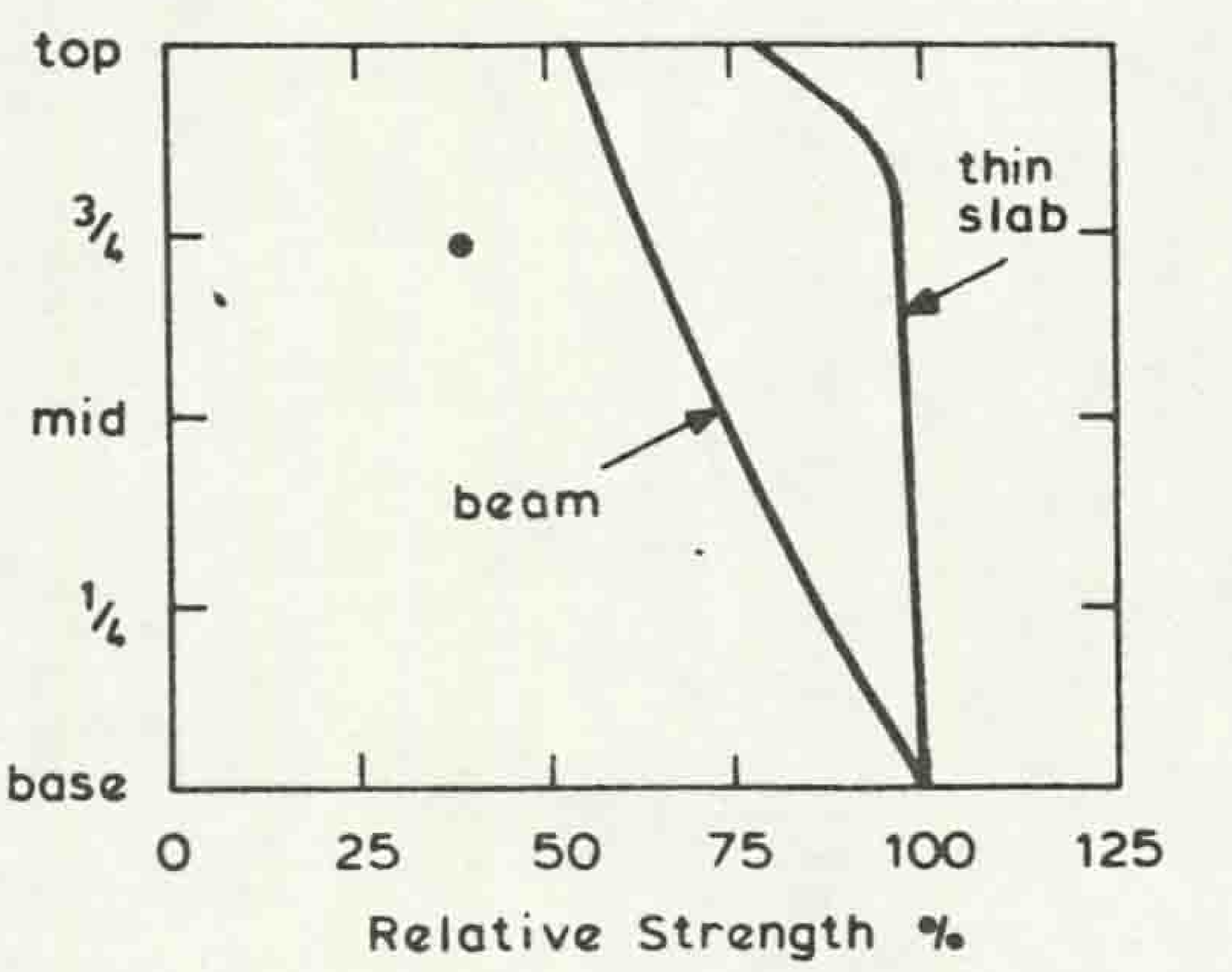

Fig. 2 -- Relative Strength Distribution Through Thickness

Field tests by the authors on a deep voided skew highway bridge deck slab which was approximately ten years old were carried out by the Windsor Probe method. Readings were taken at corresponding locations on the top surface and soffit and indicated minimum top surface strengths of 72% soffit values, but with a mean of 88%.

Strength variations with depth must thus be carefully considered when planning the location and type of in-situ testing. Surface zone tests will indicate the extreme values, but cores may not detect localised surface features and ultrasonic measurements across the depth of slabs will yield an average value. Where values are required for calculation purposes it is essential that they should relate to the critical regions of the member. If testing cannot produce this directly due to practical difficulties, the value must be estimated bearing in mind the features indicated in Figure 2. Where results are to be used for comparison with code specifications, however, it is essential that they reflect an average value of strength across the member.

Modulus of Elasticity

Whilst long-term deformation is dominated by creep and cracking, analysis of live load influences requires an estimate of the short-term elastic modulus (E_{sh}). This can be deduced from measurement of strength or dynamic modulus. Since elastic modulus is dependent both on aggregate type and proportions, the relationship to concrete strength is not clearly defined. The problems of strength estimation have been described, and it is unlikely that a value of elastic modulus could be deduced to an accuracy of better than ±15% even for an accurately known concrete strength. If a more precise value of E_{sh} is required, a value of Dynamic Modulus can be obtained from Ultrasonic pulse velocity results and then adjusted to give a corresponding value of static modulus for use in calculations.

Use of Measured Material Properties in Calculations

Design calculations use properties related to the characteristic strength of standard specimens with factors of safety. For non-linear analysis, the Model Code (CEB-FIP, 1978) recommends a stress-strain curve for loads of short duration as a function of the characteristic specimen strength and the associated value for E_{sh}. For an existing structure the relationships between in-situ and standard specimen strengths, between characteristic and mean strength, and the use of appropriate factors of safety have, therefore, to be considered.

In-situ strength may be less than that of a laboratory cured standard specimen of the same concrete due to differences in compaction and curing. The extent of this difference depends on many practical factors, but for a wet in-situ concrete in a thin slab the average 28-day equivalent cylinder strength may be as low as 65% of that of a standard specimen. For a deep slab or beam the corresponding figure would be about 90%. For a dry in-situ concrete, such as might occur under the waterproof membrane of a bridge slab, the strength may be 10% higher than for a wet concrete. However, when estimating likely future concrete strength, long-term increased strength development should not be relied upon, since it is subject to factors such as cement type, curing and environment.

Field measurements produce an estimate of the mean strength of the in-situ concrete $\bar{f}_c$ for the locations tested. The characteristic, or lowest acceptable strength, f_c, is normally assumed to be related to the mean strength by the relationship:

$$f_c = \bar{f}_c - 1.64s \qquad (1)$$

where s is the standard deviation. When insufficient test results are available to enable a direct statistical analysis, the value of s may be derived from the data presented in Figure 1, provided that testing has been concentrated on areas within the structure which are likely to give comparable strengths (e.g., top or soffit of slabs or beams). The values in Figure 1 do not encompass the strength variations with depth suggested by Figure 2, and if readings have been taken at locations likely to include such variations it will be necessary for the engineer to use his judgement to arrive at a suitable estimate of standard deviation s for use in calculations.

The value of in-situ f_c thus obtained is used for non-linear analysis and, in the light of the above comments, is likely to be a safe-side estimate. If an average E_{sh} has been obtained from testing it seems reasonable that this should be used directly for analysis. Otherwise, a value appropriate to a concrete with characteristic strength f_c should be taken from the literature.

In normal design procedures for determining the strength of sections concrete parameters are derived from the characteristic specimen cube or cylinder strength. Partial factors of safety are introduced to cover a variety of features including the difference between in-situ and control specimen strengths. Provided that the in-situ tests are located at critical regions considered in calculations, it seems reasonable to dispense with the factor which compensates for this difference. There will, however, always be uncertainties about the accuracy of test data and possible future deterioration of a structure and it is prudent to retain a factor of safety. A minimum design value of (f_c/1.2) is recommended.

NON-LINEAR FINITE ELEMENT ANALYSIS

The finite element method is too well known to be detailed here. The slab studies presented are based on a thin plate formulation (Baldwin, 1973). Stiffnesses of steel and concrete are obtained separately, with composite action being achieved through the assumption of perfect bond (Cope, Rao, 1977). Steel and concrete stresses are determined from the prevailing strain field, and assumed constitutive equations, at a grid of sampling stations over each element. In plan, these stations are located at the 2x2 Gauss integration points used to determine the element stiffness matrix and the 'released' nodal forces due to material degradation. Reinforcement is 'lumped' at these stations. The depths and directions of the outermost steel layers in each direction close to the soffit and top surface are retained, but for economy inner layers are grouped at their respective centres of gravity. Concrete stresses are determined at five equally spaced stations through the slab thickness and the appropriate Newton-Cotes formulae are used to determine integrals for in-plane forces and moments about axes in the median plane.

Loading is applied in increments and the tangent stiffness matrix is held constant over each load increment. Stiffness degradation due to material damage is simulated by comparing internally generated forces with applied loading. An iterative procedure is followed until an equilibrium position is reached to some prescribed tolerance (Cope, Rao, 1981). Using suitable constitutive equations, it is possible with this method to follow the complete load history of a slab.

For assessment of a precracked prototype structure that has been subjected to an unknown load history constitutive equations can only be approximated. It has been shown (Sparks, 1973) that for beams, the effects of fluctuating load with a sustained component are similar to those of long-term application of the loading at its maximum intensity. Although, to the authors' knowledge there is no comparable evidence for slabs, where the crack directions may be dependent on load history, it is recommended that this result be used to enable assessments to be made from analysis of a single load case.

It is proposed that the behaviour of a slab be estimated from a two-stage analysis. In the first stage, the effects of sustained and repeatedly applied loads are evaluated. The resulting model is then used as the basis for calculating response to abnormal loading and effects of structural modifications. Material models appropriate to the two phases are discussed next.

Material Property Model

Constitutive equations are based on simplified material models. These have to take into account such diverse phenomena as creep, shrinkage, bond-slip and debris in cracks.

For reinforcement, identical tri-linear uni-axial stress-strain curves are used in tension and compression. Different types of reinforcement may be used in a bridge slab and properties appropriate to each should be specified. As standards relating to steel properties evolve with time, documents contemporary with the construction period should be studied when more detailed evidence is not available.

To initiate the analysis for long-term loading, stiffness of concrete is calculated based on unstressed properties. These are, for plane stress conditions, assuming isotropy:

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{Bmatrix} = \frac{E}{(1-\nu^2)} \begin{bmatrix} 1 & \nu & 0 \\ \nu & i & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_{xy} \end{Bmatrix} \tag{2}$$

where $E = E_\ell$, estimated long-term Young's modulus; and
ν = Poisson's Ratio.

Forces mobilised by concrete are calculated using idealised, uniaxial stress-effective strain relationships (3) in principal directions.

$$\begin{Bmatrix} \varepsilon_1^* \\ \varepsilon_2^* \end{Bmatrix} = \frac{1}{(1-\nu^2)} \begin{bmatrix} 1 & \nu \\ \nu & 1 \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \end{Bmatrix} \tag{3a}$$

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \end{Bmatrix} = \begin{bmatrix} E & 0 \\ 0 & E \end{bmatrix} \begin{Bmatrix} \varepsilon_1^* \\ \varepsilon_2^* \end{Bmatrix} \tag{3b}$$

Concrete cracks when principal tensile stress exceeds tensile strength. It has been shown (Cope, Rao, 1980) that analytical response of slabs monotonically loaded to failure under laboratory conditions is sensitive to values of tensile strength and representation of tension stiffening. However, for slabs subjected to loading with a significant sustained component and repeated applications of additional multiple load patterns, there is evidence to show that tensile strength and tension stiffening are much less significant (Cope, Rao, 1981). Tension stiffening can, therefore, be neglected for current analyses, and the stress component orthogonal to cracks is set to zero. Poisson's Ratio is set to zero at a cracked station.

When stresses are evaluated in principal directions, concrete shear modulus is not used. The principal directions can be viewed as defining material property axes, with cracks crossing orthogonally to principal strain direction. Principal directions can change with redistribution induced by cracking. It is not suggested that actual crack directions rotate, but that cracks can form in more than one direction. Local concrete stiffness is most influenced by the crack whose direction is closest to the axis of principal moment for each particular load pattern. This model is clearly an approximation to actual material behaviour, but has been shown to give good predictions of short-term structural response (Cope, Rao, 1981).

Analytical Procedure

To determine the state of strain in a cracked prototype structure it is analysed with E set to an appropriate long-term value of Young's Modulus. The loading applied is the dead load plus serviceability loading applied as a u.d.l. Crack positions and inclinations are recorded and the live load component is then removed.

When equilibrium between internally mobilised forces and the permanent loading is established, the effects of abnormal loading and structural modifications can be assessed. The constitutive equations (2-3) are used, but with E set to E_{sh}, the short-term value for Young's Modulus. The uni-axial stress-strain curves used to determine additional internal mobilised forces in cracked and uncracked directions are shown in Figure 3. It can be seen that cracking is permitted in the presence of an overall compressive strain. This is usual when part of the total strain is due to time related effects. Also, a compressive stress can be carried whilst there is a total tensile strain. This is to account for the effects of bond slip and debris in cracks. At present there is insufficient experimental evidence available to thoroughly check these models, but the general approach is in accord with observed behaviour.

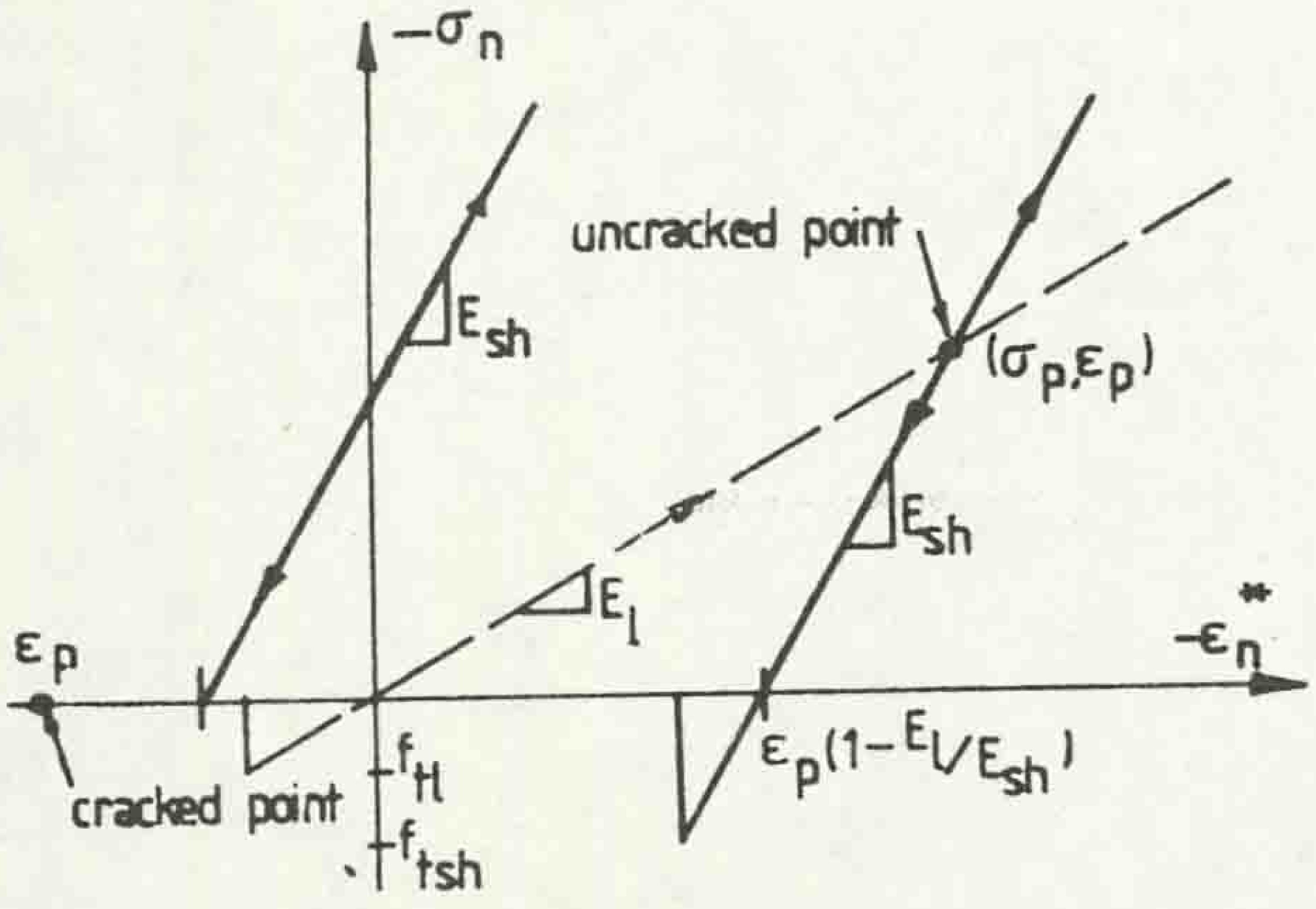

Fig. 3 -- Stress-effective Strain Curves for Long and Short-term Loading

When the effects of structural modifications, such as the addition of intermediate supports, are to be studied, the tangential stiffness matrix is modified by incorporating the support stiffness. In some cases, jacking may be used at a newly introduced support to enable it to carry part of the dead loading. This may be the case when existing bearings are overloaded or original shear stresses are dangerously high. As long-term effects are involved, a new set of values for $(\sigma_p, \varepsilon_p)$ are determined. This is done by imposing prescribed forces or displacements at the jacking points in accordance with the practice to be used on site.

An alternative approach for dealing with time effects has been proposed for beams (Scordellis, 1981) in which a further iterative cycle is introduced to allow for creep effects. This may prove to be a better approach for structures with a known load history, but to date there is insufficient experimental data to justify the additional analytical costs.

With either approach, specification of material properties is subjective and an engineer can set them to give predictions that model observable features such as deflections and cracking of prototype structures.

Material Properties to be Used for Analysis

As the values given to E_ℓ, $f_{t\ell}$, ν are necessarily subjective, it is interesting to examine their effects on predictions of behaviour of a deep prototype skew bridge slab for which self weight was the major loading. In Figure 4a the extent and directions of soffit cracking predicted for long-term loading with $E_\ell = E_{sh}/3$, $f_{t\ell} = 0.08f_c$ and $\nu = 0.2$ are shown. Analyses with E_ℓ set to values in the range $E_{sh}/3 \leq E_\ell \leq E_{sh}$ produced very similar results. Typical mid-span deflections are given in Table III. It can be seen that, with a non-linear analysis, the value of E_ℓ selected affects both the average and distribution of deflections.

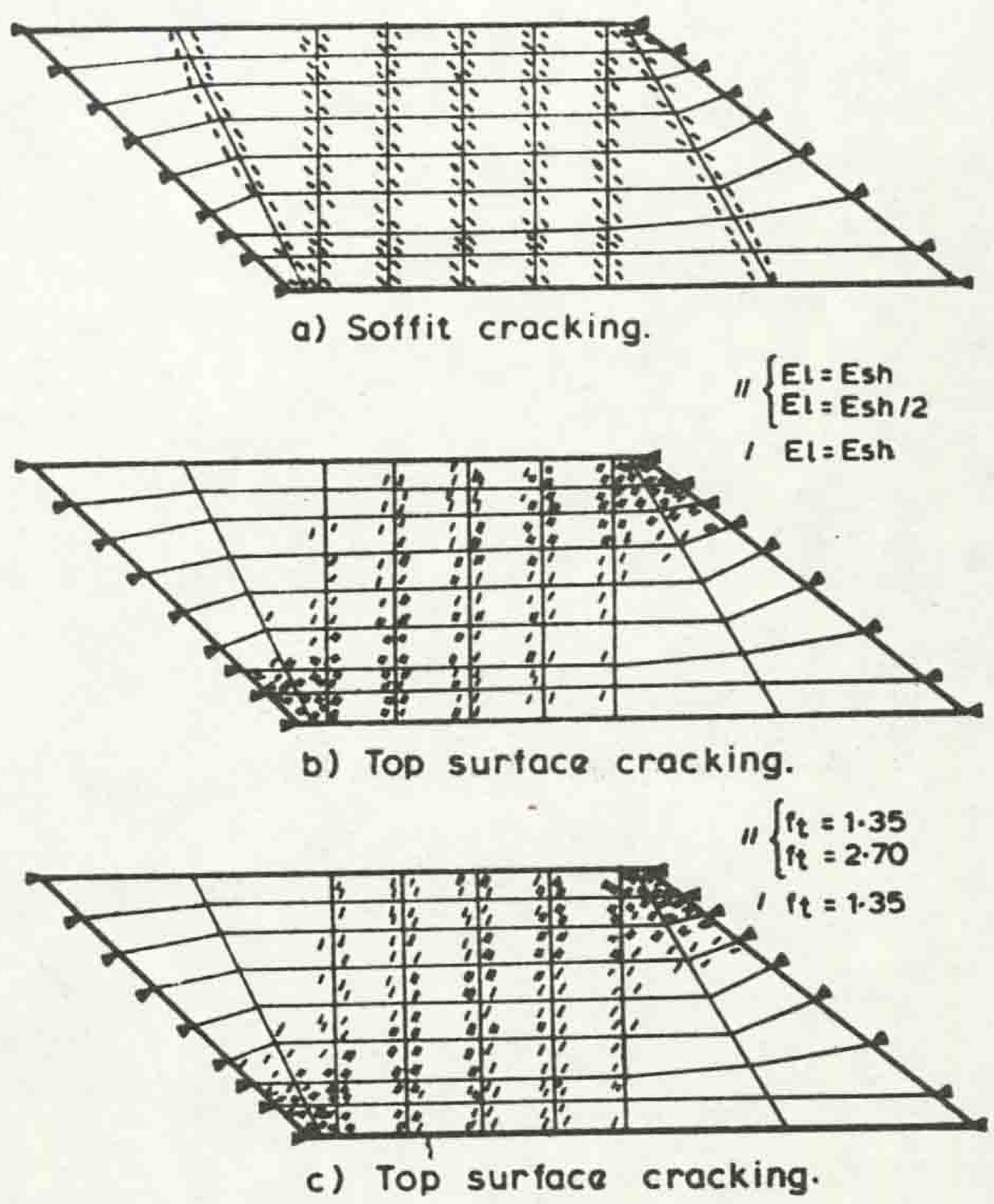

Fig. 4 -- Comparison of Predicted Crack Patterns

In Figure 4b the predicted top surface cracking for $E_\ell = E_{sh}$ is compared with that for $E_\ell = 0.5E_{sh}$. For both analyses $f_{t\ell} = 0.08f_c$ and $\nu = 0.2$. It can be seen that the spread of cracking is greater with $E_\ell = E_{sh}$. As the permanent loads are 'applied' over a short time interval (e.g., when the shutters are struck) the predictions with $E_\ell = E_{sh}$ are relevant. It is not reasonable to suppose that the cracks heal with the passing of time. To produce a crack pattern similar to that obtained with $E_\ell = E_{sh}$ but with average deflection appropriate to long-term effects from a single analysis, $f_{t\ell}$ was set to $0.04f_c$ and $E_\ell = 0.5E_{sh}$. (For this analysis the strain at which cracking is initiated is the same as for the analysis with $E_\ell = E_{sh}$ and $f_{t\ell} = 0.08f_c$).

In Figure 4c the crack patterns predicted for $f_{t\ell} = 0.08f_c$ and $f_{t\ell} = 0.04f_c$, with $E_\ell = 0.5E_{sh}$ and $\nu = 0.2$, are compared. The mid-span deflections are given in Table 3. It can be seen that the central deflections are similar. Reducing the tensile strength increases the transverse hogging curvature which leads to the desired greater spread of top surface cracking.

E_ℓ/E_{sh}	$f_{t\ell}/f_c$	Edge	Centre	Edge
1.0	0.08	129	119	161
0.75	0.08	136	127	170
0.5	0.08	149	141	185
0.33	0.08	166	161	207
0.5	0.04	161	146	202

Table III - Mid-span Deflections (mm)

Analyses performed with Poisson's Ratio in the range 0.1 to 0.2 indicated that predicted behaviour was not sensitive to the value of this parameter. Cover to reinforcement was decreased by 10% and increased by 30%, values which were considered to be extremes of allowable tolerances, but predicted behaviour was not significantly affected. Provided as-built drawings are reasonably accurate, the major parameters to be set are E_ℓ and $f_{t\ell}$. The range of values discussed above have been found to give results reasonably in accord with observed behaviour of a prototype bridge slab in service and of laboratory tests (Cope, Rao, 1981). More data on bridge behaviour has to be gathered before general recommendations on material property values can be made.

CONCLUSIONS

Non-linear finite element methods can be used to estimate the flexural response of bridge slabs in service to overload and structural modification. The material data needed for analysis of short-term effects can be obtained by non-destructive tests. Because of variation in properties, and the uncertainties involved in their measurement on site, simple material models are recommended for use in analysis.

Studies using non-destructive testing show a random variation of concrete properties in plan and an increase in strength through thickness, though possibly with a thin layer of 'work-hardened' concrete on top. At present, parameters for the analysis of long-term effects cannot be obtained objectively, and estimates may need to be adjusted to ensure that predicted deflections and crack patterns model those observed. In view of the uncertainties, it is recommended that characteristic values of properties be used at all sampling stations.

It would be useful if authorities kept a record of bridge displacements to assist with assessments.

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